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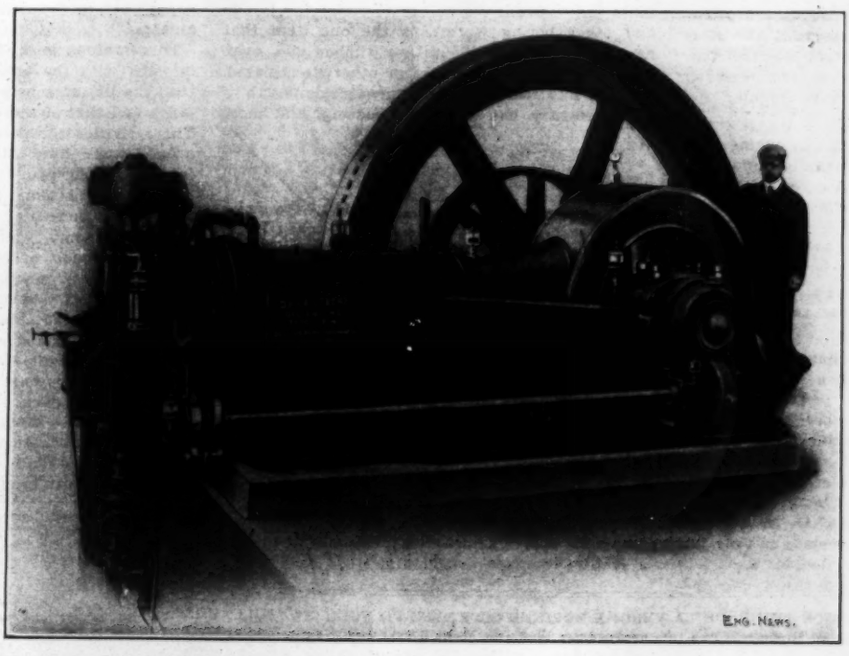
A New Oil Engine of Remarkably High Economy.

During the recent annual meeting of the American Society of Mechanical Engineers, a number of the members had the privilege of witnessing, in operation, at the works of the Le La Vergne Machine Co. in New York City, a new type of oil engine, which fully equals in thermal efficiency the famous Diesel motor and far excels that machine in practical adaptation to everyday work.

For some 14 years the De La Vergne Machine Co. has been the American manufacturer of the well-known Hornsby-Akroyd oil engine, an English invention, and many thousands of these engines have been installed. As a result of the experience with this engine and with other types of internal-combustion engine, the company has for some two years past been engaged in the development of a new type of oil engine, which should furnish more complete combustion with heavy oils than is usually obtained with the Hornsby-Akroyd, and which should secure at the same time the high thermal efficiency obtained with the Diesel engine. The design of this new machine is due to Mr. A. Franchetti, of the De La Vergne Company's engineering force. The engine is illustrated herewith.



PERSPECTIVE VIEW OF HIGH-EFFICIENCY CRUDE OIL ENGINE.

While only a half a dozen of these engines have so far been built, they have been so thoroughly tested in the De La Vergne shops that entire confidence is felt that they will prove satisfactory in regular service. The sizes now offered are from 85 to 175 HP., built with a single cylinder, and 170 to 350 HP., built with twin cylinders. This represents an advance over the sizes of the Hornsby-Akroyd engines, the largest of which was a twin-cylinder engine of 250 HP. capacity.

Referring to the fuel efficiency of these engines, the manufacturers claim a fuel consumption of about ½ lb. per brake-HP. per hour. This economy has been considerably exceeded, however, in numerous tests. Oil consumption below 0.4 lb. per HP.-hr. has been several times recorded. One test made last July by Dr. Leonard Waldo showed a consumption of only 0.374 lb. of oil per brake-HP.-hr.

A very important advantage of the engine is that the efficiency does not fall off materially with reduction of load until the load falls below 30% of the rating. The effect of this feature on the economy of the engine in daily service is obvious. Of even greater importance to the engine user than its remarkable thermal economy, however, is the fact that the engine runs satisfactorily on even the cheapest and heaviest grades of oil. Even the California crude oils, with a large percentage of asphaltum, are used in the engine with hardly a trace of color in the discharge from the exhaust pipe. Many oils have been used experimentally which were too thick to run or be fed by the feed pump at ordinary temperatures. By warming these oils, however, they became liquid enough so that they were used without difficulty.

Taking up now the mechanical features of the engine, it resembles the well-known Hornsby-Akroyd in having an externally-heated bulb or vaporizer, into which the oil is sprayed. At starting, it is necessary to warm this bulb by a blast lamp for ten or fifteen minutes. After that, the heat is maintained in it by the combustion of the oil fed into the interior at each stroke. This heat is sufficient to ignite the charge at each stroke, so that no electrical nor other form of ignition is required with the engine, and all complications with ignition apparatus are thus avoided.

The engine operates on the four-stroke cycle and uses high compression, like the Diesel engine. Instead of the enormously high compression used in the Diesel motor, however, the compression in the present engine is limited to about 200 lbs. per sq. in. The air drawn into the cylinder on the idle stroke when compressed to this extent in contact with the hot cylinder walls and piston reaches a temperature, probably, of between 700° and 800°. The charge of oil is sprayed through this hot compressed air into the interior of the vaporizing bulb by a jet of compressed air compressed to a higher pressure, in which again this engine resembles the Diesel motor. The pressure of the compressed air used for spraying, however, is only some 600 lbs. per sq. in., as compared with the 1,000 lbs. or so which the Diesel motor requires. This lower pressure of compressed air is possible because of the lower compression in the cylinder. It will be seen that the margin between the air pressure in the cylinder, and the pressure of the compressed air used for spraying, is about 400 lbs. per sq. in. in each case.

The amount of compressed air used in spraying is very small, and the air compressor is not a separate machine but is a very small two-stage direct-acting compressor, attached to the side of the cylinder of the main engine and driven by an eccentric on the main shaft.

The fuel oil is delivered to the air spray nozzle by a pump, and the governing of the engine is effected by varying the length of stroke of this pump through the action of the governor. This latter is of a standard type, and it operates to move the fulcrum of the lever by which the pump is operated. In this way a large variation can be made in the length of the stroke, without any reaction on the governor, and the engine runs quietly with very little change in speed when the load is thrown entirely off.

The reason for the small variation in economy with large variations in load is probably found in the fact that complete combustion takes place, whether the engine is running at full load or at a quarter-load or less. The same volume of air is drawn in on the forward stroke, and compressed, in each case. When the engine is running at full load, all this air is mixed with the atomized oil and burned. When the engine is running at much reduced load, however, the amount of air is the same, and the oil

simply takes up enough air to complete its combustion; the remaining air acts simply as a cushion, and returns to the engine on the working stroke practically the same work that was absorbed in its compression.

The air compressor used for spraying the oil is also utilized for starting the engine. The air compressed by the first stage of the compressor goes to a reservoir, and the second stage of the compressor draws from this reservoir. To start the engine, a simple hand lever on the rear of the cylinder admits air from the reservoir and turns the engine over until regular explosions begin.

The combustion obtained with this engine is so complete that it is not necessary to remove the vaporizer for cleaning at all. A defect of some of the earlier oil engines was the clogging of the tiny needle valve which admitted the oil to the cylinder. In the present engine, however, the needle is a steel rod of substantial construction and as thick as a man's finger, so there is very little opportunity for the spraying passage to become clogged.

While the engine herewith illustrated and described has been designed with a special view to the practical requirements of power users, and simplicity in construction has been a chief aim, it is noteworthy, from a scientific point of view, as representing the highest attainments yet reached, probably, in any type of heat engine, in the economical conversion of heat into power. The accompanying table shows the results of a series of tests upon this engine made in July last by Dr. Leonard Waldo, M. Am. Soc. M. E.

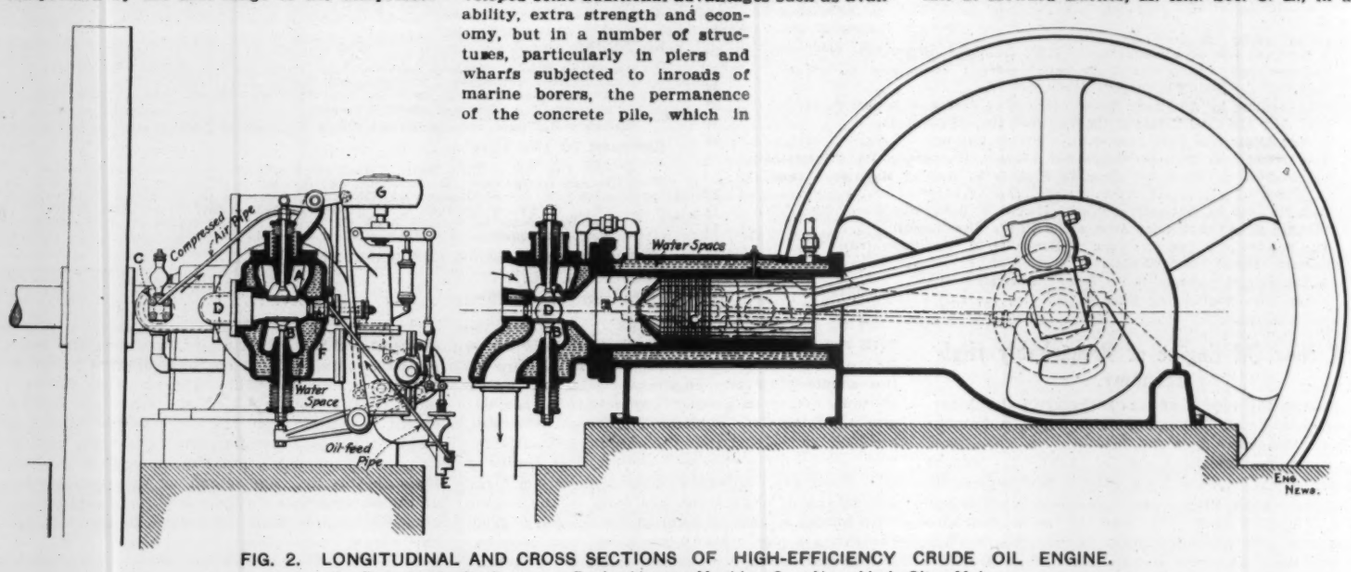


FIG. 2. LONGITUDINAL AND CROSS SECTIONS OF HIGH-EFFICIENCY CRUDE OIL ENGINE.
A. Franchetti, Designer, De La Vergne Machine Co., New York City, Makers.

A Inlet Valve. B Exhaust Valve. C Two-Stage Air-Compressor. D Vaporizer. E Oil-Pump. F Oil Inlet. G Governor.

Concrete Shell Casings for Protecting Wooden Piles Against the Teredo.

The concrete pile is the result of the very evident necessity for some foundation support or underpinning more permanent than the long used wooden pile. As it has been improved it has developed some additional advantages such as availability, extra strength and economy, but in a number of structures, particularly in piers and wharfs subjected to inroads of marine borers, the permanence of the concrete pile, which in these cases means its ability to resist the teredo and other boring worms, is the one item that has made it preferable to any timber pile, even when the latter is creosoted or otherwise treated. In some locations, however, the extreme length of pile necessary has made the casting, and more especially the handling, of a concrete pile most difficult, and a construction has been devised in which an untreated wooden pile is sheathed with a shell of concrete, or sometimes tile, and the annular space filled in with concrete. This gives an easily handled unit, stronger than a wooden column and quite thoroughly protected against destruction by marine borers. One of the best illustrations of this type of work is a wharf of the United Fruit Co. at Bocas del Toro, Panama, recently described by its supervising engineer, Mr. T. Howard Barnes, M. Am. Soc. C. E., in a paper* before the American Society of Civil Engineers.

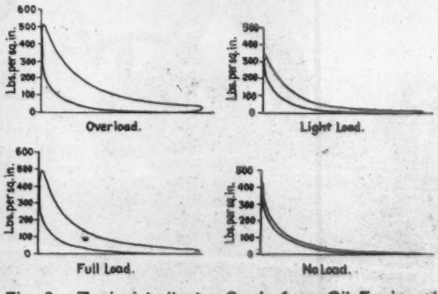


Fig. 3. Typical Indicator Cards from Oil Engine at Full Loads and at Light Loads.

This Panama port, like most tropical harbors, is infested with the teredo, and it had been found that the life of a non-treated pile was about five years and that of a creosoted pile about 15 years. The activities of this animal, while most marked around water-line, extend down the whole wetted length of the pile to a few inches below the shifting sea bottom. In rebuilding its fruit-loading wharf, therefore, the company, in the interests of economy, was forced to design the subaqueous portion of some teredo-resisting material, preferably concrete. The depth of the longest pile needed was such that a 70-ft. length was required, and as this length introduced difficulties of handling and manufacture, with the semi-skilled labor available, it was decided to use for the under structure of the wharf untreated southern pine piles, encased with concrete shells extending some distance below the sea bottom, with the annular space between the pile and the shell filled in with a lean concrete.

At first it was intended to make these shells in

*The Reinforced-Concrete Wharf of the United Fruit Co. at Bocas del Toro, Panama; by T. Howard Barnes, M. Am. Soc. C. E. Proceedings of the American Society of Civil Engineers, Sept., 1900, p. 901.

RESULTS OF TESTS MADE ON 125-HP. DE LA VERGNE MACHINE CO.'S TYPE FH FUEL OIL AND CRUDE OIL ENGINE BY DR. LEONARD WALDO.
(Duration of Trial, 10 hours. Two Oils used: Solar Fuel Oil, about 0.8600 Spec. Grav. and California Crude Oil, about 0.953 Spec. Grav.)

	1	2	3	4	5	6	7	8	9	10	Average of four-hour runs on Solar fuel oil.	Average of four-hour runs on Cal. crude oil
Amperes	312	312	313	308	212.5	222	269	305.5	305	301.5	311	295
Volts	261	259	258	257	202.5	202	264	264	268	267.5	259	268.3
KW. switchboard	81.6	81	80.9	79.5	43	44.9	76.6	80.5	82	80.6	80.5	79.9
Electrical HP.	109.3	108.5	108.5	106.5	57.5	60.2	102.5	107.7	109.8	108	——	——
B. HP., 84% eff. gen.	130	129	129	126.5	68.5	71.6	122	128.2	130.5	128.5	128.5	127.3
R. P. M.	156	156.5	157	157	158	156	156	156	156	156	156	156
Temperature cooling water inlet, F.	73°	74°	74.5°	74.5°	74.5°	74°	74°	74°	74°	74°	——	——
Temperature cooling water outlet, F.	176°	185°	189°	163°	148°	146°	172°	184°	171°	182°	——	——
Consumption cooling water, gallons.	292	289	288	409	326	xxx	330	386	350.8	xxx	——	——
Cooling water, per B. HP., hour.	2.25	2.24	2.23	3.23	4.76	xxx	2.70	3.03	2.74	xxx	2.84	2.82
Spec. grav. fuel oil, Beaume.	32°	32°	33.6°	33.5°	33.5°	33°	17°	15°	18°	18°	——	——
Temperature fuel oil, Fahr.	86°	88°	92°	94°	94°	94°	90°	122°	116°	116°	——	——
Con. per hr. fuel oil, lbs.	53.2	54.6	48.2	54.2	33.3	32.9	55.8	64.1	67	58.2	——	——
Con. per B. HP., hour.	0.408	0.422	0.374	0.427	0.487	0.458	0.465	0.505	0.514	0.453	0.408	0.484
Lub. oil, lbs. per hour.	3.47*	3.47*	1.95	1.95	xxx	1.25	0.937	0.75	0.75	xxx	——	——
Cyl. oil, lbs. per hour.	1.125	1.125	1.0	1.0	xxx.	0.885	0.75	0.937	1.11	xxx	——	——
Lub. oil, lbs. per B. HP., hour.	0.026	0.027	0.0153	0.015	xxx	0.017	0.007	0.006	0.006	xxx	0.021	0.006
Cyl. oil, lbs. per B. HP., hour.	0.0096	0.0097	0.0077	0.008	xxx	0.012	0.0063	0.007	0.008	xxx	0.009	0.007
Temp. resistance, tank, cent.	xxx	xxx	xxx	38°	38°	37°	35°	36°	36°	36°	——	——

*Excess due to oil leakage.

EXHAUST GAS ANALYSIS.

	1	2	3	4	5
	No Load.	Full Load.			Half Load.
CO^2	1.0	5.7	5.9	6.0	4.0
O	18.8	13.0	12.5	12.3	15.7

4-ft. lengths and to place them around the pile by the aid of a diver, but they were finally built as a single shell of tapering sides and dropped around the already driven wooden pile. The shells protruded some distance above the wooden pile, and the space between them as well as the upper totally hollow space was filled with concrete, lean at the bottom and of a regular 1:2:4

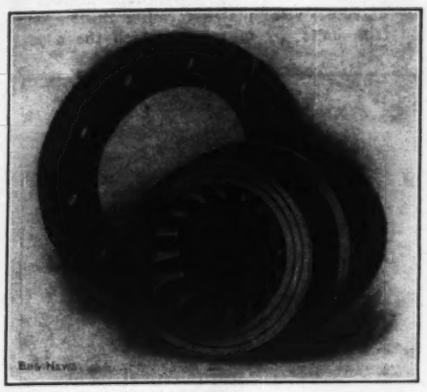


Fig. 4. Vaporizer Cap for Oil Engine.

mixture at the top. In this upper portion vertical reinforcing rods were set, and upon the top of each pile was placed a concrete column which made a part of the main framework for the reinforced-concrete structure of the wharf.

The construction of the wharf proper entailed no particular difficulty, except that due to the very poor grade of labor procurable, nor does its design present any great novelty, but from the paper noted we give below the portions relating to the pile-shell manufacture and placing.

SHELL FORMS.—The 4-ft. shells, or rather pipes, which were at first planned to be used for pile protection, were never made, but, as stated previously, single shells long enough to reach from above high water to below the sea bottom were substituted, and these were reinforced so as to stand handling and placing by the pile-driver.

Soundings were taken at each pile location, and the shells were made of sufficient length to permit their sealing into the sea bottom and to reach above ordinary high water. Furthermore, the shells were made and stacked in the order in which they were to be used, thus greatly facilitating their placing.

The attempts to utilize adjustable forms and home-wound wire reinforcement need not be described; but it should be noted that such forms must be very firm and stiff, and capable of preserving the desired thickness of shell, qualities which adjustable forms do not possess; and that it is too difficult to adjust any closed wire cage over the core. The forms adopted leave improvements to be made. They consisted of five parts: two half cylinders of No. 16 gage galvanized iron, a spider or spacer of wood, a wooden cradle lined with sheet metal, and a wooden arch faced with sheet metal.

The half cylinders—really not cylinders but frustums of a cone—when fixed over the spider formed the core of the shell. They tapered from 20 ins. to 16 ins. in diameter. The cradle rested horizontally on a truck, and, together with its companion arch, formed an outer mold having a diameter 4 ins. greater than the corresponding core, thus giving a thickness of shell of 2 ins.

The exact length of shell desired was secured by stops, in the form of collars, made of short sections of wood attached to sheet metal. They served the further purpose of keeping the core concentric with the outer form. The forms were made up in lengths of 16, 18, 20, 24, 28, 30 and 32 ft.; there were several of each length, excepting of the longest ones.

The reinforcement of the shells was, for the most part, the electrically-welded fabric made by the Clinton Co. The mesh best adapted, and the one principally used, was 6 × 6-in. with No. 6 longitudinal and No. 8 roundabout wires. Some few home-made spirally-wound cages were made for shell reinforcement, but they proved to be expensive and were difficult to use.

SHELL MAKING.—The stone ingredient of the concrete was necessarily limited to a size not exceeding ¾-in. in diameter. By careful grading, a mixture yielding 13 cu. ft. of concrete to the barrel of cement was obtained. The order of making was first to place the proper thickness of concrete along the bottom of the cradle, and therein embed the reinforcement. The latter opened longitudinally, and was wired together when the next step was taken, that of setting the core in place. After closing the wire about the core and clamping down the latter, the concrete was filled in to the "springing line" of what constituted the arch of the shell as it lay in a horizontal position. The arch form was then clamped on, and the remaining concrete was placed, openings along the top of the arch form having been left for this purpose. The concrete was compacted by churning it into place with light flat iron bars and by tapping the outer molds. This method was effective in expelling the entrained air, and made a very solid concrete. Where in contact with the forms, the concrete was left very smooth, and the only additional finish was a wash of grout. The open spaces at the top were necessarily left to be finished by hand. The forms were slightly oiled to assist in the removal of the shell.

The freshly molded shells were carefully rolled along the track a minimum distance, and allowed to stand over night.

SHELL CURING.—While the concreting was going on, two other gangs were at work, in addition to those mixing and placing the concrete. One small gang was preparing the cores and placing the stops, and a larger gang, equal to that required at the molding, was busy unloading and stacking the shells. The concrete set very rapidly, so that all shells excepting those 25 ft. or more in length could be unloaded on the storage runs the morning after they were made. The spider was drawn first, and then the core forms, the upper one of the latter requiring ordinarily tedious cutting away of the fine rich concrete which had oozed into the lap of the upper over the lower portion of this form. Then the top or arch form was removed and the cradle was rocked on its side, thus throwing the shell out on the run. Here it was inspected and patched, the latter being required in but few instances, and was kept moist for several days (Providence greatly assisting). The runs consisted of two lines of supports along the shells at their quarter points. A slight inclination toward the water facilitated the movement of the shells on the runs.

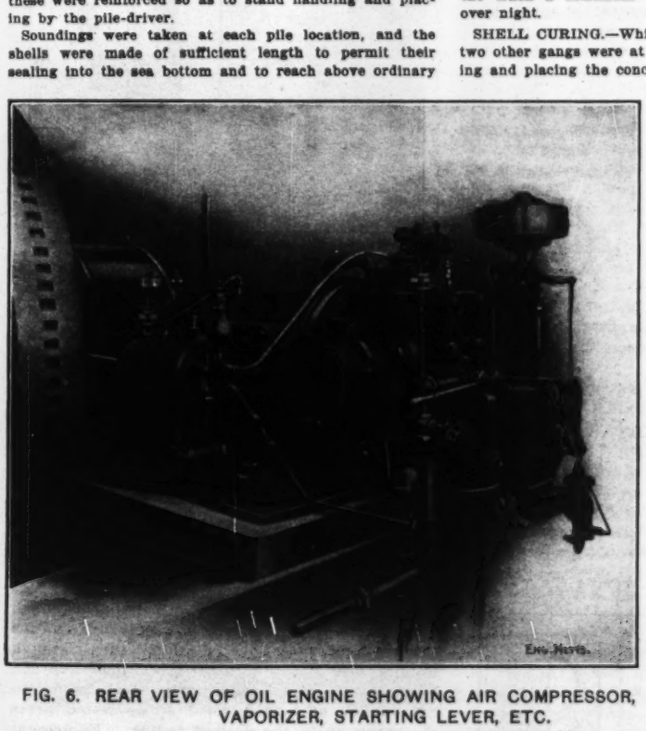


FIG. 6. REAR VIEW OF OIL ENGINE SHOWING AIR COMPRESSOR, VAPORIZER, STARTING LEVER, ETC.

As stated, most of the shells were less than 24 hours old when they were unloaded; the longer ones were a day older, yet so rapidly does concrete set in an air temperature ranging from 75° to 90° F. in the full 24 hours, that the total losses from all causes amounted to only 2%. Each shell was marked with a letter and number indicating its position.

The description of this process of producing the shells sounds simple in its recital, and so indeed it is, but the forms were heavy and the preparation was somewhat tedious; the men needed constant watching and spurring on to their task, and the result was that, with delays occasioned by using the molding force to hurry up the preparation of the forms, or to unload some of the shells, it was possible to produce regularly not more than six shells per day. Altogether, the production of the shells was the hardest portion of the entire work.

SHELL PLACING.—The placing of the shells proved to be very easy. A special box punt, decked over, was used for the conveyance of the shells. This was tied

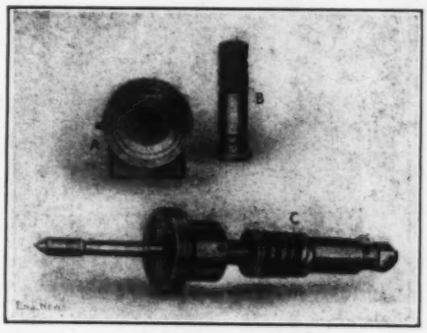


Fig. 5. Parts of Oil Spraying Nozzle for Oil Engine.

broadside at the foot of the run, and the shell was rolled thereon over short skids. The punt was towed by rowboat to the pile-driver, and the shell was lifted by the fall with a rope sling into vertical position and placed about the pile just driven. The shell was allowed to drop with a slight momentum in order to bed the toe, and any further settling needed was secured by resting the hammer and frame on the shell top, or by slight drops of the hammer, with a cushion of rope and plank interposed. In some cases the shell was not long enough indicating a settlement of the coral about the pile. In these cases, a sheet-metal casing was clamped about the head of the shell, and, usually with some difficulty, the head was extended to the proper grade. The time consumed in placing the shell was about six-tenths of that used in driving the pile.

SHELL FILLING.—After wedging the shell firmly in a concentric position about the pile head, a sufficient quantity of fine, rich concrete was deposited quietly within to seal about 2 ft. in depth at the foot of the shell. After two days or more it was generally possible to pump out the enclosed water, and then, under dry conditions, the lean concrete could be placed. This mixture was carried to within 5 ft. of the top, where the column reinforcement was embedded. The concrete for this portion was made of the rich mixture. Then the pile with its concrete envelope was jacked and clamped into position ready for the forms of the superstructure.

There were in all 432 shells in the wharf, averaging 18.4 ft. in length; in the shells there was 267 cu. yds. of concrete and in the annular filling 229 cu. yds. Working with very incompetent labor the cost per linear foot of shell was $1.78. The average wages paid to these laborers was $1.20 gold for 9 hours work and Mr. Barnes says that "in point of service rendered the $1.00 man was worth, say, 40% of the value of an ordinary Italian (not an experienced one)."

As a final word of caution to anyone contemplating such construction in tropical waters, the author says:

In construction of this kind care should be taken to extend the protective shell to a depth secure against any future corrosion and consequent exposure of the wood. Recent examples of the destruction of even creosoted piles by sea life at depths of 25 ft. emphasize this caution.

DANGER TO CARS FROM HIGH-TENSION TROLLEY wire was the subject of some tests recently by German Government railway officials in conjunction with the Allgemeine Elektricitäts Gesellschaft at Berlin. Using 6,000 volts on the trolley wire, a car was slowly run under a section of the wire that hung down low enough to touch the car. An arc formed on the wires touching the wood, but it went out in a few seconds as the circuit-breaker in the power-house was opened by the overload. When the car roof was covered with roofing-paper, the contact of the wire caused no perceptible leakage, even though the roof was moist. When the wire came in contact with metal parts of the roof or car body, a short-circuit arc was formed, but all current took its path over the metal sheathing without danger to the inside of the car. It was concluded that running steam trains over electrically equipped sections of track involves no danger to passengers (Schweizerische Bauzeitung, Dec. 25).

Tapering Concrete Chimneys.

The majority of the numerous reinforced-concrete chimneys that have been built are cylindrical in shape, due to limitations in the design of the forms, and where a variation in diameter is made this has been effected by an offset. There are some objections to this offset arrangement (partly on account of the interruption of the vertical steel reinforcement), and also to the

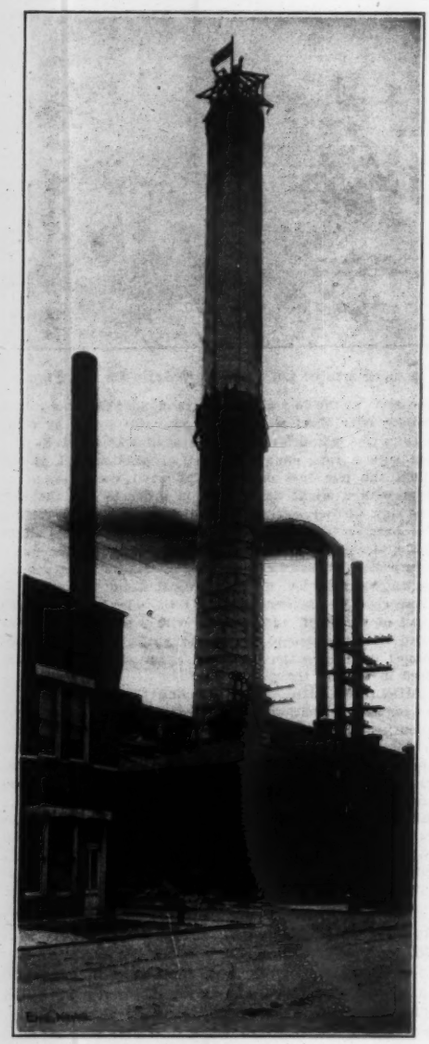


Fig. 1. Tapering Concrete Chimney at Muncie, Ind.
(Showing the treatment of the surface with a cement wash.)

appearance of tall stacks of uniform diameter. It is practicable of course to build chimneys tapering by means of special forms, but the extent and expense of forms of the ordinary character required for any one chimney is too great for commercial use. Recently, however, some special designs of form construction have been devised which permit the building of tapering stacks without excessive cost for this feature of the work, and two examples of such construction are described below.

TAPERING CHIMNEY AT CHILLICOTHE, O.—In Figs. 2 and 3 are shown the appearance and general design of a tapering or "coniform" concrete chimney built by the Weber Chimney Co., of Chicago, by the use of the Weber unit form, which is a patented design. The form consists of two rings or courses (used alternately), each composed of vertical wooden units so arranged that the required number of units can be changed during the progress of the construction of the chimney so as to form the required taper in its height. For convenience in handling, the forms are put together in sections. Fig. 5 shows the handling of these sections, with the removal of the lower ring of the form and its replacement on top of the upper ring. The removing and replacing of each course of the form (including the inside and outside shells) occupied about three hours, and after the concrete was deposited it was allowed to set for 18 hours before removing the form.

This chimney (which is the largest yet built under this system) is at the plant of the Mead Pulp & Paper Co., at Chillicothe, O. It is 175 ft. high above the ground, with an interior diameter of 14 ft. 3¾ ins. at the base and 9 ft. at the top. It has a uniform reduction in diameter equivalent to 1 ft. in diameter for each 28 ft. in height, and the thickness is reduced ⅛-in. for each 3½ ft. in height (or for each course of the forms). The thickness of the concrete wall is from 11⅞ ins. at the base to 5 ins. at the top.

The steel reinforcement consists of vertical bars which take up all tension caused by wind pressure; the maximum tension allowed on the steel is 16,000 lbs., and no allowance is made for tension in the concrete. The maximum compression allowed is 500 lbs. per sq. in. Horizontal rings of ½-in. round bars are placed 14 ins. apart and are outside of the vertical bars. The arrangement of the vertical bars is shown in Fig. 3. In the determination of wind stresses in this chimney the company's engineers used the system described by Prof. E. R. Maurer (University of Wisconsin) in our issue of Sept. 10, 1908.

TAPERING CHIMNEY AT MUNCIE, IND.—A large tapering chimney which is of special interest from its structural design as well as from its tapered form is shown in Figs. 1 and 4. This has been built recently at the plant of the Muncie Electric Co., at Muncie, Ind. The American Gas & Electric Co., of New York (which controls the Muncie company), entertained proposals for a self-supporting steel chimney (lined to the top with fire-brick), and a tapering reinforced-concrete chimney. In conjunction with the engineers, Sargent & Lundy (Chicago), the company elected to use the latter type of construction.

This construction of a tapered wall, keeping both inside and outside sections perfectly circular and concentric, and getting the amount of batter or taper evenly distributed on each side of the center line, was carried out by the use of special steel forms, which are the invention of Mr. Carl Stieler. The contract was awarded to the General Concrete Construction Co., of Chicago, which owns this special form design.

The forms, which are 5 ft. high, consist of sets of rings, steel plates forming the surfaces and long bolts by means of which the adjust-

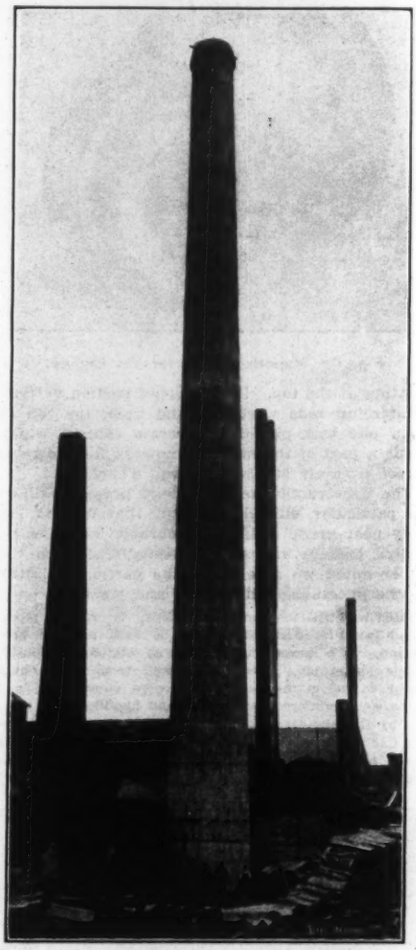


Fig. 2. Tapering Concrete Chimney at Chillicothe, O.

ments of diameter and wall thickness are made. The rings are of sufficient diameter to enclose the base of the chimney, and are made of heavy angles bent to varying radii and spliced into complete circles. The radial bolts must be long enough to permit of their extending in to the chimney surface when the diameter is smallest (at the top of the chimney). The diameter of the rings remains constant, standard sets of form rings being carried in stock for various sizes of chimneys. The steel sheets are of standard size, and are filled out with split sheets, made interchangeable. The sheets are secured by flat-head stove bolts to vertical stiffeners on the outer surfaces of the forms, except at one rib, where the adjustment is taken up. The varying diameter is obtained by means of the long radial bolts which turn through small angles attached to the steel rings, and end in malleable clips which grip steel bands on the outside of kerfed wood strips which press the sheets in to the desired radius. Permanent centers are kept at the base of the chimney and on the scaffold above, and each section of the chimney is plumbed. Thus perfect alinement is secured throughout the construction.

This chimney is of particular interest, as the engineers specified that it should be designed to withstand a wind velocity of not less than

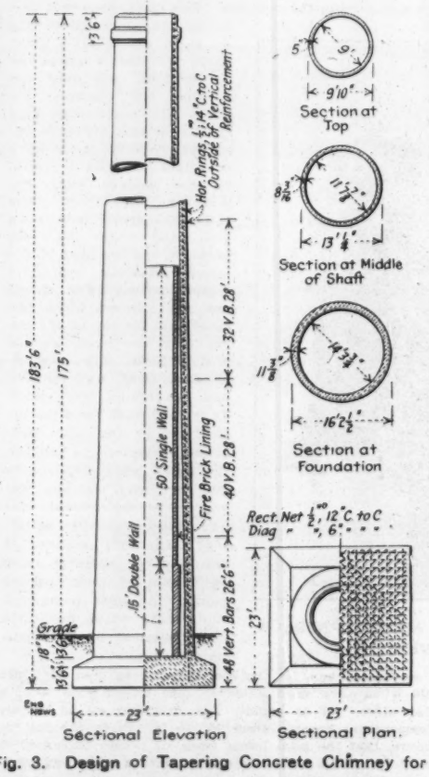


Fig. 3. Design of Tapering Concrete Chimney for the Mead Paper & Pulp Co., at Chillicothe, O.
(Weber Chimney Co., Chicago; Builders.)

100 mi. per hr., and that it should be of sufficient weight to overcome the wind moment resulting from such a wind and have no tension in the concrete or the steel reinforcement on the windward side. It is usual practice in the design of reinforced-concrete chimneys to stress the vertical steel reinforcement to about 16,000 lbs. per sq. in., the steel taking all of the tension and the concrete being counted upon for compression only. In this case, however, the chimney walls were made sufficiently heavy as to necessitate no reinforcement except for temperature and shrinkage cracks. The chimney is actually a "gravity" design, closely following the practice in brick chimneys.

The lining is carried the entire height of the shaft, as shown. It consists of one course of fire-brick, backed for 33% of its height with two courses of hard common brick, and then with one course. The lining is separated from the concrete shell by an air space which varies from about 24 ins. to 30 ins., owing to the brick lining being vertical and the concrete shell tapering. The breeching connections consist of two openings on opposite sides of the chimney, the gases from which enter the chimney and strike the baffle wall set at an angle of 45°.

The chimney is 252 ft. 6 ins. high, with an inside diameter of 15 ft. 6 ins. or 14 ft. 6 ins. inside the lining. The shell is 20⅝ ins. thick at the base, tapering uniformly to 8 ins. at the top. The vertical reinforcement consists of ⅝-in. square twisted bars about 24 ins. apart, surrounded from top to bottom with sheets of triangular wire mesh consisting of ¼-in. round rods spaced 4 ins. vertically and laced triangularly with lighter wire. These sheets are wrapped horizontally around the vertical steel on the outside, and are placed within 2½ ins. of the outside surface of the chimney, thus preventing surface cracks. Around the breeching openings particular attention was paid to the reinforcement to carry the weight of concrete over these openings.

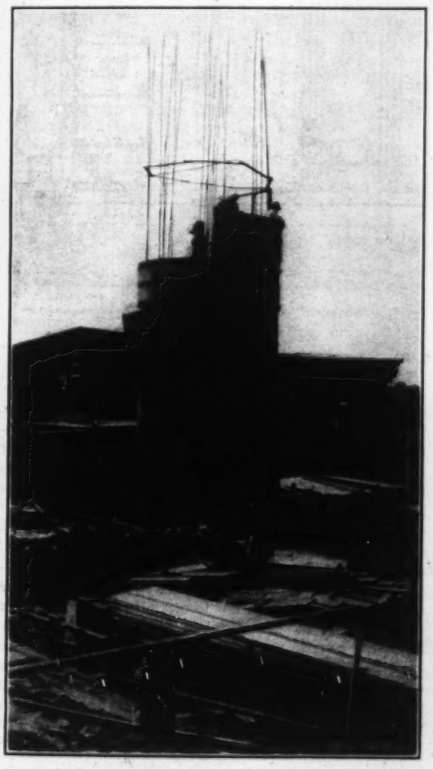


Fig. 5. Handling Sections of Weber Unit Forms for Tapering Concrete Chimney.

The chimney rests upon a rectangular foundation 38 × 42 ft., built in the shape of a truncated pyramid 3 ft. high on a slab 2½ ft. high. The foundation rests upon a bed of hard gravel capable of bearing many times the load imposed. The foundation is about 20 ft. below the grade line and directly above it is the basement of the boiler room. On one side of the foundation rests a heavy brick wall separating the engine room and boiler room, and this is the reason for the oblong shape.

The concrete in the chimney proper is composed of 1 part Portland cement to 2½ parts sand and 3 parts of gravel not exceeding 1-in. diameter. This mixture was made wet enough so that when the steel forms were filled with the mixture, water would float to the top. In addition to this wet mixing, tamping was resorted to, thus insuring perfect bond between the steel and concrete, and the filling of voids.

For the foundation, a concrete composed of 1 part Portland cement, 3 parts sand and 6 parts of crushed limestone was used. Stone was permissible as the foundation of the chimney is not subjected to heat with its possible bad effects upon limestone. This foundation was reinforced with two nets of ¾-in. square twisted bars. A net running parallel with the sides of the foundation was placed within two or three inches of the top of the 2½-ft. slab, and another net of bars placed at an angle of 45° to each side of the foundation and within 3 ins. of the bottom of the slab. The bars are spaced 24 ins. in the upper and 12 ins. in the lower net. The vertical bars from the chimney extend down into the foundation and are bent under the diagonal net at the bottom of the foundation, thus providing anchorage in addition to the bond with the concrete.

After starting the shaft and getting it above the breeching connections, progress was made at the rate of 5 ft. per day, very few days being lost. Some difficulty was experienced when the construction reached the height of the old steel stacks alongside, as the wind carried the smoke and gases into the faces of the men at work, necessitating a delay of a day or two. The delay from this cause was small.

After completing the chimney and its lining, the outside was washed from top to bottom with a cement wash containing sufficient lime to whiten the surface, and the sunken panels or flutes in the head were painted black to give contrast. This wash effectually removed form marks and streaks resulting from the slopping over of the extremely wet concrete, and greatly improved the appearance. Fig. 1 shows the rigging used in the process of washing, the platform being raised or lowered by ropes over the chain blocks hung from the cross-tree built on the top of the scaffold inside and extending above the top of the chimney. No outside scaffolding was used, the entire work being done from a braced tower built of sticks 4 × 4 ins. in sections, on the inside of the chimney as work progressed. All hoisting of mixed concrete and materials was also done from the inside, obviating danger to workmen employed on the other parts of the power plant.

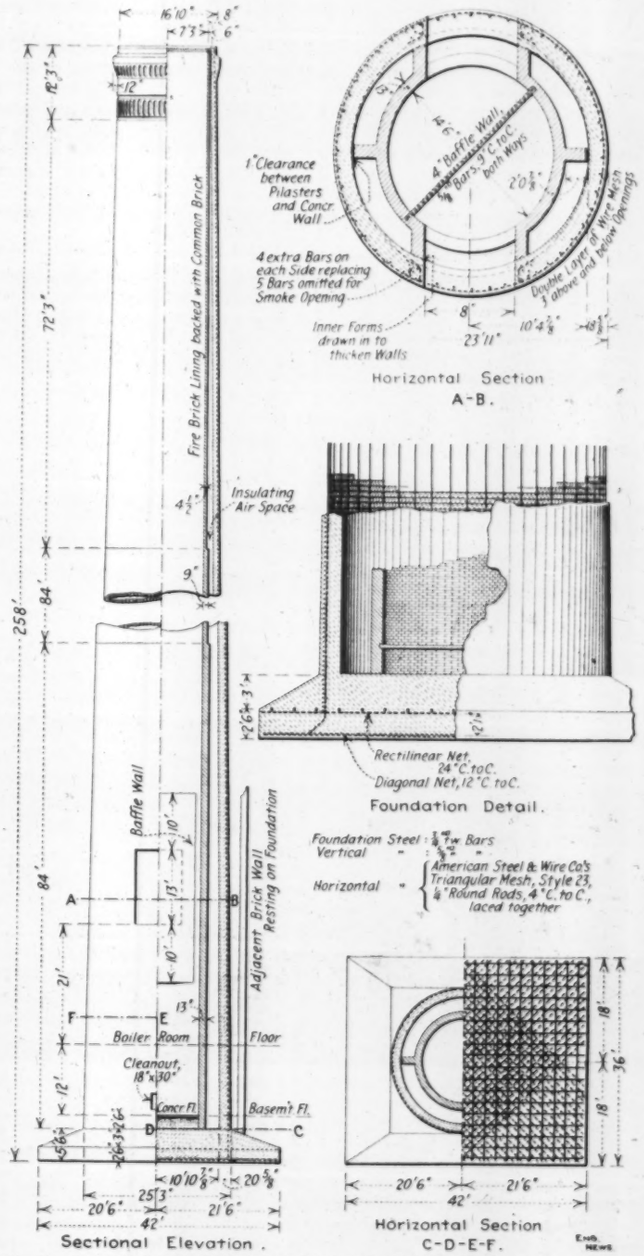

Fire Brick Lining backed with Common Brick
Insulating Air Space
Baffle Wall
Adjacent Brick Wall Resting on Foundation
Boiler Room Floor
Cleanout, 18"x30"
Concr Fl
Basem't Fl
Sectional Elevation.
1" Clearance between Pilasters and Concr Wall
4 extra Bars on each Side replacing 5 Bars omitted for Smoke Opening
Inner Forms drawn in to thicken Walls
Horizontal Section A-B.
Rectilinear Net, 24" C. to C.
Diagonal Net, 12" C. to C.
Foundation Detail.
Foundation Steel: Vertical
Horizontal
American Steel & Wire Co's Triangular Mesh, Style 23, ¼" Round Rods, 4" C. to C., laced together
Horizontal Section C-D-E-F.

FIG. 4. DESIGN OF TAPERING CONCRETE CHIMNEY OF THE GRAVITY TYPE FOR THE MUNCIE ELECTRIC CO., AT MUNCIE, IND.
General Concrete Construction Co., Chicago; Builders.

ELIMINATION OF ALL CROSSINGS AT GRADE is being gradually effected by the Pennsylvania R. R. Since Jan. 1, 1900, according to information sent out by the Pennsylvania Co., 673 grade crossings have been removed on the lines of this company east of Pittsburg and Erie. This includes only work completed up to Sept. 1, 1909, and does not include the ten crossings to be eliminated by the projected change of the line through Bristol, Pa., on the New York division. On the lines between New York and Washington, Philadelphia and Pittsburg, 256 public grade crossings and 129 private crossings at grade have been removed in the past ten years, leaving only 774 public crossings scattered over 574 miles of road. On the division between Philadelphia and Washington alone, 106 grade crossings have been eliminated; between Jersey City and Philadelphia, 78 crossings have been removed; and, on the Pittsburg division, 46 crossings have been taken out.

Coal Cars and Coal Trains of the Virginian Ry.

The Virginian Ry. is a new line which is of more than ordinary interest in that it was built specially for the transportation of coal from the mining districts of West Virginia to the Atlantic seaboard. For this reason it was built as a direct line with low grades against the direction of heavy traffic, in order to attain a high degree of economy in operation by handling the main coal traffic in very heavy trains. These conditions involved much heavy and costly work in the construction of a low-grade line through mountainous and broken country.

The railway was opened for its entire length in April, 1909. It is 441 miles long, from Deepwater, W. Va. (where connection is made with the Chesapeake & Ohio Ry.), to Sewalls Point, Va. (on Chesapeake Bay), where extensive freight terminals and shipping piers have been established. There is a branch into Norfolk, Va., for passenger and general freight service. The maximum grades against the heavy eastbound traffic are 0.2%, compensated for curvature, except at two points where pusher grades are required. One of these has a grade of 2.07% for 12½ miles near Princeton, W. Va. (the starting point of the coal trains), and the other has a grade of 0.6% for 10 miles at Whitehorn, W. Va. The maximum grades against westbound traffic are 1.5%.

In view of the special purpose of the railway, considerable attention was given to the question of the freight train equipment, for the purpose of obtaining high power and capacity with low cost for maintenance and repairs. The locomotives and cars therefore were built largely to special designs which were prepared by Mr. R. P. C. Sanderson, Superintendent of Motive Power, and we are indebted to him for plans and other information.

Coal Cars.

A large proportion of the freight equipment consists of 50-ton steel gondola cars, 43 ft. long over the couplers and weighing 21.65 tons empty. These cars were designed specially with a view to their use in connection with Hulett car-dumping machines, as all the coal delivered over the shipping piers at Norfolk is discharged from the cars by the use of these machines. For this reason it was practicable to use a long flat-bottom gondola car, but it was necessary to make the sides specially strong to prevent bulging under the stresses produced when the cars are rolled over, and also to prevent injury to the edges of the sides by the clamps which hold the cars in position on the cradle when being dumped. When the loaded car is secured in the cradle the truck springs are compressed by the weight, and as soon as the load is discharged by the tilting of the car, these springs (which cannot raise the empty body on account of its being clamped in the cradle) exert a pressure which tends to cripple or buckle the sides unless it is properly provided for.

The leading ideas involved in the design of the coal car may be stated as follows: (1) suitability for handling by the unloading machines, (2) low center of gravity, (3) great strength to withstand shocks in train service and yard work, (4) ease of repair, (5) standard details. This last idea was for the purpose of using as many of the same details as possible in the design of underframes for other equipment, including not only box, flat, coke, stock and other freight cars, but also cabooses and tenders. The idea has been carried out very thoroughly, and will result in a low cost of maintenance, as well as in a considerable saving in the amount of car repair material to be kept in stock.

On account of the great length and weight of the trains and the great power of the locomotives used in hauling them (as well as the power of the switching engines which push the trains up the inclines of the gravity yards), exceptional strength has been given to the center sills, draft attachments and couplers. The details of these parts were developed by a careful series of tests, and the cars fully meet the requirements of strength without having excessive weight or dead load.

The general design of the coal cars is shown in Fig. 1, and some of the details are given in Fig. 2. The principal longitudinal member of the underframe is a box girder formed by a pair of center sills (12⅞ ins. apart) and a 20-in. top cover plate. Each sill is a 15-in. 40-lb. rolled steel channel with the flanges outward, and having an angle riveted to the inside at the bottom. This member is intended mainly to take care of the buffing and draft stresses, the load weight being carried by the sides. The body bolsters also are box girders, each formed of a pair of pressed steel channels (of varying depth) with the flanges outward and having top and bottom cover plates. At three intermediate points are transoms or transverse diaphragms, similar to the bolsters, but having a single channel and a bottom plate; while at four points there are shallower pressed steel diaphragms of uniform depth. Floor plates ¼-in. thick are fitted against each side of the cover plate of the central girder, and are riveted to the transverse members. Their outer ends are flanged upward and riveted to the side plates. The end sills are pressed steel channels with flanges outward.

The sides are designed as plate girders of ample strength to carry the entire load. The ¼-in. side plates extend to the bottom of the transverse members, and are riveted to their ends, while the lower edge is reinforced by a longitudinal inside angle. Pressed steel stiffeners of U section are riveted to the outside at intervals of 3 ft. 3 ins., and these are reinforced on the inside by brackets riveted to the floor plate and side plate (and stakes). They are connected at the top by a steel bulb angle having its vertical leg on the inside of the side plate, while the horizontal leg fits over the stakes. This angle forms not only a stiffening rib longitudinally, but forms the bearing of the side of the car against the dumping cradle when the car is tilted on its side. It thus strengthens the side against buckling under the unusual lateral stresses set up by this method of handling. Inside stake pockets are provided for temporary tall stakes when the car is loaded with lumber, the load rising above the sides.

The ends of the car are of similar construction, but the plates extend only to the level of the floor (being riveted to the upturned flanges of the floor plates and to the back of the end sill. They have inside stiffeners of flanged gusset plates resting on the central girder or sill, and two outside horizontal Z-bar stiffeners; one of these latter carries a bracket for the brake staff.

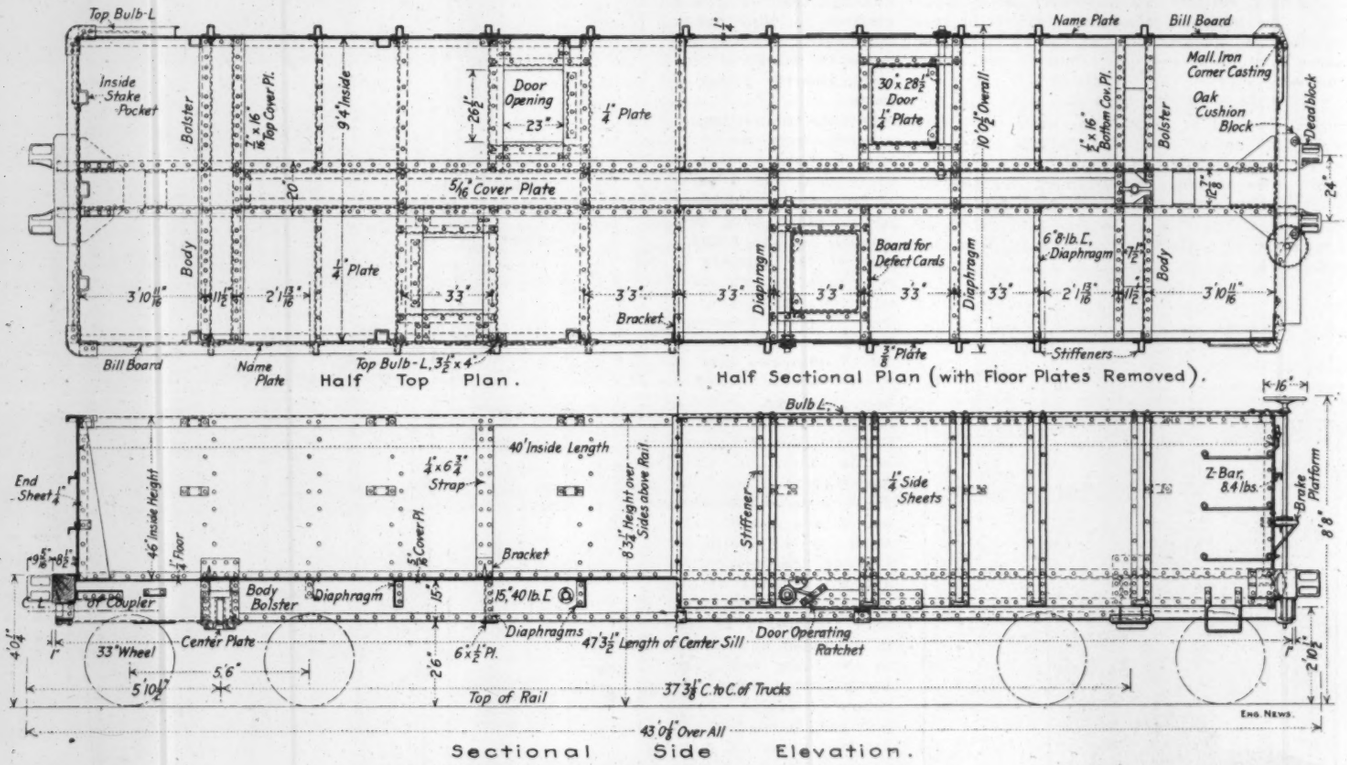


FIG. 1. STANDARD 50-TON STEEL COAL CAR; VIRGINIAN RY.
R. P. C. Sanderson, Superintendent of Motive Power.

While the unloading is ordinarily done by the car dumping machines, provision is made for unloading at coaling stations and in local business by means of bottom openings, having doors flush with the floor of the car. There are four of these doors, with openings 26¼ x 28 ins. in the clear. They are arranged in staggered positions, by an inside block, and a broad gusset plate is riveted over the center sill and against the back of the end sill, while a clamp fitted over the bottom flanges of the center sill prevents any spreading or crippling at this point. To the face of each end sill is fitted an oak cushion block 8 x 10 ins., to which are bolted two malleable-iron dead-

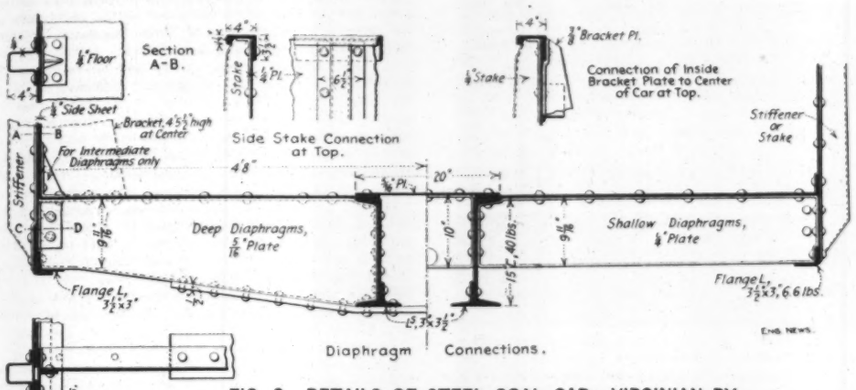

Section A-B.
Floor
Side Sheet
Bracket, 4 5½ high at Center
For Intermediate Diaphragms only
Stiffener
Side Stake Connection at Top.
Stake
Bracket Pl.
Connection of Inside Bracket Plate to Center of Car at Top.
Stiffener or Stake
Deep Diaphragms, ⅝" Plate
Shallow Diaphragms, ½" Plate
Flange L, 3½ x 3½ x 3
Flange L, 3½ x 3½ x 6.6 lbs.
Diaphragm Connections.
Section C-D.
L, 3½ x 3½ x 6.6 lbs.

FIG. 2. DETAILS OF STEEL COAL CAR; VIRGINIAN RY.

so that there is always more than half of the lateral strength of the floor plates to prevent distortion in the event of collision or derailment. In cars of this type on some roads the doors are in pairs (across the car) so that the transverse strength of the floor is greatly reduced, with the result that cars are often buckled and bent at these points.

blocks. The draft rigging is of the Farlow-Westinghouse combination, and is so made that the coupler cannot be driven in to its full limit until after the deadblocks have come in contact and have compressed the oak cushion blocks.

The riveting is in general of ⅝-in. rivets in 11/16-in. holes, except that ⅞-in. rivets are used in the draft gear and ¾-in. rivets for the bolster cover plate. In assembling, all metal faces in contact are heavily coated with red-lead lute before being riveted together. The outside is cleaned with wire brushes and given three coats of paint; the first is Japinole, applied by spraying; the second is standard brown oxide paint, and the third is a black sealing coat of lamp-black with a lead base.

The trucks, Fig. 3, are of the diamond-frame type, with ⅞-in. pressed steel bolsters. A special feature is that provision is made for the lubrication of both the center bearing and side bearings. The former has a cast steel top center-plate and drop-forged bottom plate, with annular bearings; and the top or body plate has oil holes. The truck side bearing is renewable a riveted casting having formed in it a grease cup which is closed on the outside by a screw plug and has a hole opening at the face of the bearing. With the car empty, the normal clearance of the side bearings is 1 in. The cars have 33-in. 750-lb. cast-iron wheels of the M. C. B. standard.

The principal dimensions of the coal cars are given in Table I.:

TABLE I.—STANDARD STEEL COAL CARS: VIRGINIAN RY.

	Ft.	Ins.
Length inside	40	3
Length over center sill	41	3½
Length over dead-blocks	43	0½
Width inside	9	4
Width over side stiffeners (or stakes)	10	0½
Height inside	4	6
Height from rail to top of side	8	3¼
Trucks, wheelbase	5	6
Trucks, distance between centers	31	3⅝
Wheels, diameter	2	9
Axle journals	5½ x 10 ins.	
Weight, total	43,300 lbs.	
Load capacity (rated)	50 tons	
Actual average loading	55 tons	

An important auxiliary to the coal traffic is the coke traffic, as the railway serves a large coking district. The coke is handled in 40-ton capacity stock cars which are 40 ft. long over the end sills.

Coal Train Locomotives.

For the main-line coal train service there are three types of locomotives: (1) the road engines, which are of the 2-8-2 class; (2) the pusher engines on the two grades of 2% and 0.6%, which are of the Mallet duplex type with a 2-6-6-0 wheel arrangement, having two groups of driving wheels and a leading pony truck; (3) the switching engines, which are of the 0-8-0 class. These last are required to be of sufficient power to handle the full train loads hauled over the road by the 2-8-2 engines; they are used in the general yard service, and in pushing the trains over the humps of the gravity yards.

All important details requiring renewal in service are made interchangeable for the 0-8-0 and 2-8-2 engines, thus reducing the cost of repair parts in stock and facilitating the repair work. The engines have a number of special features intended to ensure strength and low cost of maintenance; they were built to the designs of Mr. Sanderson.

The principal dimensions of the three classes of engines are given in Table II. Engines of the 2-8-2 class are used in gathering the cars from the mines and bringing them to the main coal yards at Princeton, W. Va. Lighter engines of this class are used in local service.

Coal Trains.

The standard coal train on the main line consists of 80 cars, with nominally 4,000 tons of coal, but with an allowance of 10% overload this will be 4,400 tons of paying load. The weight of the 80 cars is about 1,732 tons, making 6,132 tons behind the tender, while the engine (2-8-2), tender and caboose (18 tons) increase the

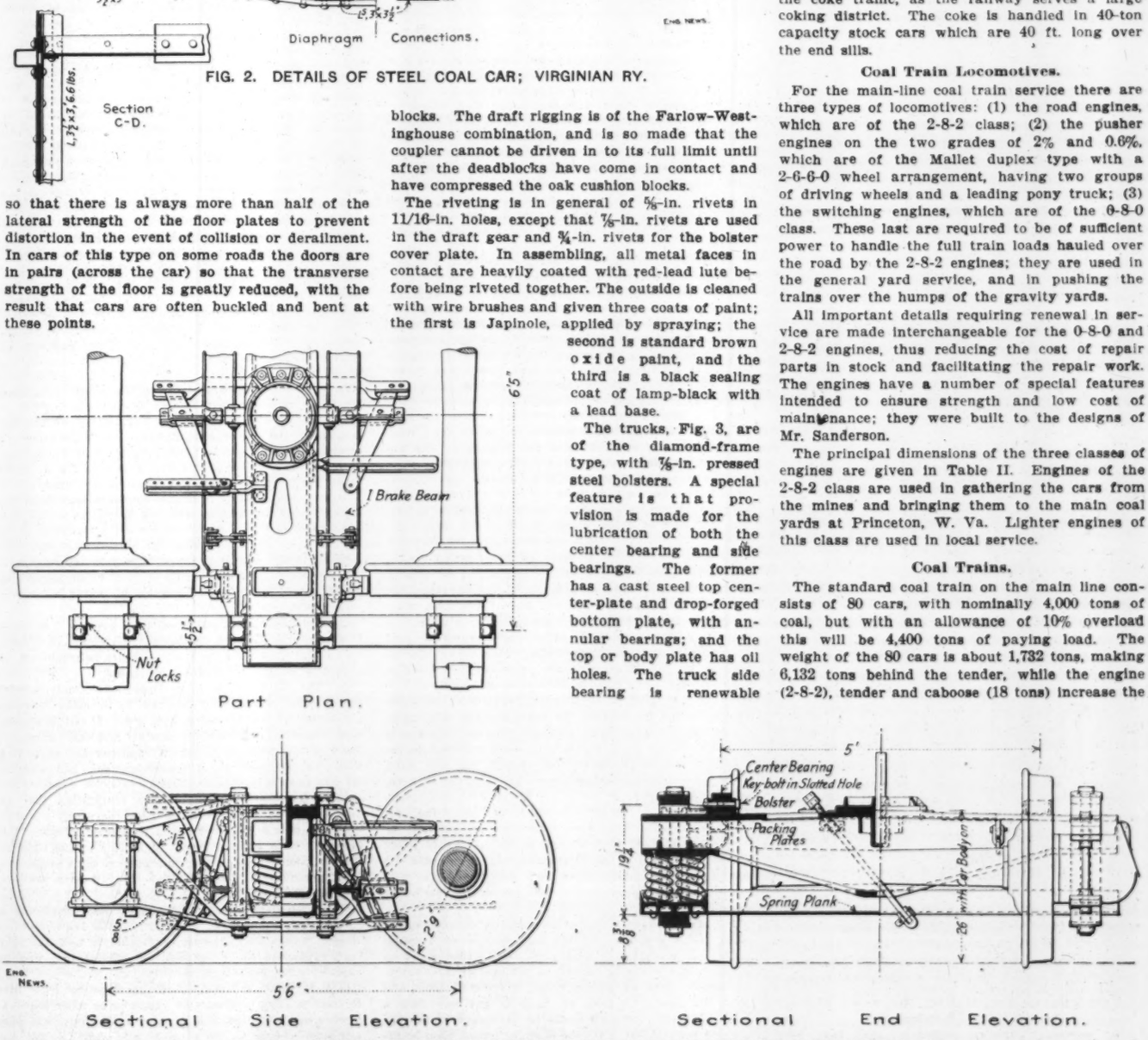


FIG. 3. STANDARD TRUCK FOR 50-TON COAL AND FREIGHT CARS; VIRGINIAN RY.

The steel doors are rectangular flanged plates, with malleable iron hinge attachments riveted to them. Each door is operated by a pair of chains on a shaft fitted with a ratchet and crank handle.

The drawbar opening in the end sill is reinforced cast-iron block with chilled top face 4 x 8 ins. This block is fitted to a pocket on the bolster and secured by a cotter bolt; the bolt hole in the block is slotted so that the height of the bearing can be adjusted by packing plates in the bottom of the pocket. The body side bearing is

total weight of the train to 6,368 tons. This weight is exceeded occasionally. The heaviest train that has been handled was composed of 100 standard steel coal cars with 5,500 tons of coal, and the total weight of this train was as follows: cars and coal, 7,562 tons; caboose, 18

TABLE II.—DIMENSIONS OF COAL TRAFFIC LOCOMOTIVES: VIRGINIAN RY.

	Road.	Pusher.	Switching
Service			
Class	2-8-2	2-6-6-0	0-8-0
Driving wheels	4 ft. 8 ins.	4 ft. 6 ins.	4 ft. 3 ins.
Truck wheels, front	2 ft. 6 ins.	2 ft. 6 ins.	.....
Truck wheels, rear	3 ft. 0 ins.	.....	.....
Wheelbase, driving	15 ft. 6 ins.	37 ft. 9 ins.	14 ft. 0 ins.
Wheelbase, rigid	15 ft. 6 ins.	11 ft. 0 ins.	14 ft. 0 ins.
Wheelbase, total	31 ft. 9 ins.	39 ft. 11 ins.	14 ft. 0 ins.
Length over engine and tender	73 ft. 8¼ ins.	80 ft. 3⅜ ins.	61 ft. 2⅝ ins.
Weight on drivers	219,000 lbs.	312,000 lbs.	182,300 lbs.
Weight, total	269,000 lbs.	330,000 lbs.	182,300 lbs.
Cylinders, h. p.	24 × 32	22 × 30	22 × 28
Cylinders, l. p.	.....	35 × 30	.....
Valve gear	Walschaerts.	Walschaerts.	Stephenson.
Boiler, diameter	6 ft. 6¾ ins.	6 ft. 4 ins.	5 ft. 10 ins.
Boiler pressure	200 lbs.	200 lbs.	200 lbs.
Firebox, type	Radial stay	Radial stay	Radial stay
Firebox, length inside	8 ft. 6 ins.	9 ft. 6 ins.	9 ft.
Firebox, width inside	6 ft. 0 ins.	6 ft. 0 ins.	3 ft. 6 ins.
Firebox, firebrick arch	Yes	Yes	No
Tubes, No.	373	390	354
Tubes, diameter	2¼	2¼ ins.	2 ins.
Tubes, length	19½ ft.	21 ft.	15 ft.
Heating surface, tubes	4,300 sq. ft.	4,840 sq. ft.	2,763 sq. ft.
Heating surface, firebox	198 sq. ft.	200 sq. ft.	117 sq. ft.
Heating surface, arch tubes	30 sq. ft.	24 sq. ft.	.....
Heating surface, total	4,528 sq. ft.	5,064 sq. ft.	2,940 sq. ft.
Grate area	51 sq. ft.	57 sq. ft.	31½ sq. ft.
Tender, wheels	2 ft. 9 ins.	2 ft. 9 ins.	2 ft. 9 ins.
Tender, weight loaded	173,000 lbs.	173,000 lbs.	111,400 lbs.
Tender, water in tank	9,500 gals.	9,500 gals.	5,000 gals.
Tender, coal in bunker	14 tons	14 tons.	10 tons
Tractive effort	50,350 lbs.	70,800 lbs.	45,200 lbs.

tons; engine, 135 tons; tender, 83 tons; total 7,798 tons. This immense train was hauled from Victoria to Sewall's Point, Va., 125 miles, in 8 hrs. 42 mins., or at a rate of nearly 15 mi. per hr. (including three stops for water and two at passing sidings). Engines are changed twice in the run of 346 miles from Princeton to Sewall's Point. A much heavier train has been hauled with a Mallet locomotive as described in a note on p. 55 of this issue.

A New Explosives Ordinance for the City of Chicago.

The combustion of dynamite was a leading cause of the disastrous effects of a fire at Chicago last spring, the investigation attributing these effects largely to the rapid combustion, the intense heat and the suffocating fumes due to the dynamite stored in the building. This fire was at the crib (in Lake Michigan) of the working shaft of the new 73d St. water tunnel, and was described in our issues of Jan. 28 and Feb. 25, 1909. The design for the new crib superstructure, of fire-resisting construction, was shown in our issue of June 17, 1909.

The coroner appointed three engineers upon the jury called by him to investigate the accident, and as a result of the jury's report the city authorities appointed a special commission to prepare an ordinance regulating the sale and use of explosives. This ordinance was passed at a meeting of the Chicago city council on Dec. 20, 1909. Among its special features it requires the licensing of all persons and firms selling or keeping explosives, and the certification of all persons or employees having to do with the handling and use of explosives.

In view of the importance of the ordinance, and the strict character of its provisions, we give it below substantially in full, except for the elimination of some of the legal verbiage and duplication. The commission was composed of the following members, the first three of whom were members also of the coroner's jury: A. Bement, Chairman (consulting engineer); Benzette Williams (consulting engineer), John M. Ewen (consulting engineer), Arthur E. Paul (chemist of Mariner & Hoskins), C. R. Cave (merchant), Edward C. Rice (Austin Powder Co.), James Horan (Fire Marshal), John J. Hanberg (Commissioner of Public Works), John Ericson (City Engineer), G. A. Allen (Aetna Powder Co.), Edward J. Brundage (Corporation Counsel), W. F. Dunn (Alderman), Michael Zimmer (Alderman).

An Ordinance Regulating the Sale, Storage and Use of Explosives.

(1) No person, firm or corporation shall have, keep, sell, use or give away any gunpowder, guncotton, blasting powder, giant powder, dynamite, nitroglycerine, detonators, electric detonators, fulminate of mercury, or any other high explosive, or any substance or compound or mixture or article having properties of such a character that alone or in combination or contiguity with other substances or compounds may decompose suddenly and generate sufficient heat or gas or pressure or all of them to produce rapid flaming combustion or administer a destructive blow to certain persons or things, or keep, sell or offer for sale or transport or give away any loaded paper or metallic shot shells or cartridges, or percussion caps or primers, within the corporate limits of the City of Chicago, and in the waters of Lake Michigan, including all breakwaters, piers and permanent and temporary structures therein for a distance of three miles from the shore between the north and south lines of the city extended, excepting in the manner and upon the conditions herein provided and under license issued by the City Clerk on the recommendation of the Chief of the Fire Department. Said Chief of the Fire Department shall have the power to revoke the license in case in his judgment there is an infraction of the provisions of this ordinance.

(2) Any person, firm or corporation desiring a license to keep, sell or offer for sale or give away any gunpowder, guncotton, giant powder, dynamite, nitroglycerine, detonators, electric detonators, fulminate of mercury or any other high explosive of whatsoever nature, or loaded paper or metallic shot shells or percussion caps or primers, or detonators, shall make application in writing to the Chief of the Fire Department for a license so to do. The application shall set forth the name, residence and occupation of the applicant, if an individual; and if a corporation, its name, principal place of business and the name and residence of its officers. Such application shall also set forth the location at which it is desired or intended to keep such high explosives or cartridges, etc., and the maximum amount (or number) intended to be kept on hand at any one time. And whether such explosives are to be kept in bulk or in barrels, canisters or other containers.

(3) Upon receipt of such application, the fire marshal shall make or cause to be made an investigation for the purpose of ascertaining whether the place is so situated that a license to keep such explosives or cartridges, or caps, or primers, in the quantity desired would not be so dangerous as to constitute a nuisance or be a menace to the safety of the public or of adjoining property. And also whether the conditions under which such high explosives or cartridges, etc., are to be kept or handled shall be such as to provide the maximum of safety.

(4) If the result of such investigation shall be satisfactory to the fire marshal he shall approve such application and transmit same with his approval thereon to the City Clerk. Upon payment by such applicant to the City Collector of a license fee of $25, the city clerk shall issue a license authorizing such applicant to keep, sell, or give away, at the place designated, gunpowder, guncotton, giant powder, dynamite, nitroglycerine, detonators, electric detonators, fulminate of mercury, or other high explosives of similar nature, or loaded paper or metallic shells or cartridges designed or intended for use in shotguns, pistols, rifles, or other firearms, or percussion caps or primers, for and during the period of such license.

Provided, however, that if any person or corporation shall desire a license for the sole purpose of keeping, selling, or giving away within the city, loaded paper or metallic shot shells or cartridges designed or intended to be used in shotguns, pistols, rifles, or other firearms, and shall not desire to have on hand at any one time a greater quantity of such shells or cartridges in the aggregate than 25,000, a license shall be issued upon application as hereinbefore provided, and such license shall be issued upon payment of a fee of $10.

(5) All contractors and others now engaged in (or purposing hereafter to engage in) any blasting operations shall make application in writing, upon blank forms to be furnished by the Fire Department, to the Chief of the Fire Department for a license to keep and use high explosives. This application must give name, location of office or place of business, the nature and the site of the work to be performed, the intended location of the magazine or magazines, and the quantity and kind of explosives to be kept therein.

(6) Previous to the issue of licenses as herein prescribed, applicants shall furnish and file with the Chief of the Fire Department a bond approved by the City Comptroller, conditioned for the payment of any loss, damage or injury resulting to persons or property by reason of the use, sale or keeping of such high explosives, and for the strict observance of the ordinances of the City of Chicago and of these regulations. Said bonds shall be in amounts as follows:

(A) For manufacturers, agents and all others who desire to bring to or sell within the corporate limits of the city high explosives, a bond in the penal sum of $5,000; and in case of delivery being by wagons an additional sum of $5,000 for each and every wagon in excess of one wagon engaged within the city in the delivery of said high explosives as provided herein.

(B) For all contractors or others now engaged in or purposing hereafter to engage in any blasting operations, a bond in one of the following classes shall be required: (1) For the right to use or have on hand in any one day high explosives not exceeding 50 lbs., a bond in the sum of $5,000. (2) Not exceeding 100 lbs., not less than $5,000 or more than $10,000. (3) Not exceeding 200 lbs., $7,500 to $15,000. (4) Not exceeding 400 lbs., $7,500 to $20,000. (5) Not exceeding 2,500 lbs., a bond for not less than $10,000 or more than $25,000.

(7) Before any operation shall begin under a license for the transportation or for the use of high explosives in blasting, as herein provided, the applicant shall designate in writing the name or names of the person or persons whom he has designated to load holes or discharge high explosives, to prepare charges and load the holes, to transport by wagon or otherwise, or to have the care of magazines as herein provided. All such persons before being permitted to exercise any of such functions shall make an application in writing to the Chief of the Fire Department for a certificate of fitness, and before the issuance of the same shall be examined and found to possess the qualifications hereinafter specified in order that they may properly fill such positions under the regulations herein prescribed. And no person shall be permitted to have the actual care and handling of such high explosives without having first obtained a certificate of fitness as herein provided. These certificates shall be subject to inspection at all times. For the issuance or renewal of any such certificate an annual fee of $5 shall be collected by the City Clerk.

To receive a certificate of fitness the person must have the following requirements: (A) Be at least 21 years of age. (B) Be able to read and understand the English language. (C) Be of good habits. (D) Have letter of recommendation from last two employers, if he has not been in the service of his last employer for at least three years, in which case a letter testifying to good character and capacity from last employer shall be satisfactory. (E) Be familiar with the provisions of the law and the regulations governing the transportation, storage and use of high explosives, particularly that part relating to the service to be performed by the applicant. (F) Be familiar with the risks incident to the service to be performed by him, and capable of taking all necessary precaution.

(8) Nothing herein contained shall prevent a licensee from applying for and obtaining a certificate of fitness. The actual work done must at all times be conducted by a person holding a certificate of fitness.

(9) The Chief of the Fire Department shall appoint an inspector whose duty it shall be to make frequent inspection of all premises and work of all licensees. Said inspection shall include detailed and exact examinations of the manner in which licensees are complying with the requirements of the regulations, and whether all due and reasonable precautions to avoid accidents are being taken, and shall include a verification that all employees who are performing work for which a certificate of fitness is required are in possession of same. The inspector shall make a report in writing to the Chief of the Fire Department at the close of each day's inspection, stating conditions observed, and these reports shall be kept on file.

(10) The Commissioner of Health shall designate a chemist employed in the laboratory of the Department of Health, whose duty it shall be to analyze and test such high explosives as are brought or submitted to him by applicants for licenses or permits.

(11) It shall be the duty of the inspector from time to time to bring a sample of dynamite or other high explosive manufactured, sold or used by licensees and submit such sample to the chemist, who shall analyze and test same. If it shall be found that such sample is not of standard purity and quality, such license shall be revoked by the Fire Chief.

(12) All dynamite containing nitroglycerine shall be tested by the chemist according to the potassium iodide starch heat test, otherwise known as the British heat test.

(13) No license shall be granted to any person, firm or corporation engaged in the manufacture, use or sale of

dynamite or other high explosive, until such manufacturer, dealer, contractor or such other person heretofore referred to, shall first submit a sample of such dynamite or any other high explosive sought to be manufactured, used or sold, to the chemist for analysis. Whenever such chemist shall approve that such sample is of standard purity, a license shall issue if all other requirements heretofore made are complied with.

(14) No high explosives shall be transported by wagon through the streets of the city unless said wagon is in charge of a competent person holding a certificate of fitness, which certificates shall be open to inspection by any officer of the Fire or Police Department. Said certificate shall be issued only to employees of corporations or firms or persons duly licensed to transport or sell high explosives in the city.

A wagon used for transportation of high explosives shall be in good condition for service. It shall have springs and a wooden top extending over bed, enclosed on both sides and back; an additional floor laid lengthwise of sound material not exceeding 3 ins. wide, tongued and grooved, planed smooth and fair, and blind nailed.

No metal tools or other pieces of metal shall be carried within the wagon except in a separate tool box. No percussion caps or detonators, or other combustible material shall be transported in the same vehicle with high explosives.

A wagon carrying high explosives shall be drawn by a horse or horses amply able to draw the load and avoid stoppages other than to load and unload, and no unnecessary stops or stands shall be made. No high explosives shall be left in a wagon unless said wagon is in charge of an employee with a certificate of fitness, and a driver shall not leave unattended a wagon loaded with high explosives.

A wagon carrying high explosives shall have painted on it in easily legible letters at least 6 ins. high: "Dangerous—High Explosives," on sides and back. It shall have also the owner's name displayed on each side so as to be read easily at a distance of 100 ft., and in case of a licensee having more than one wagon, these shall be plainly lettered: "Wagon No. 1," or "Wagon No. 2," etc.

A wagon carrying high explosives shall avoid, whenever possible, those streets on which there is a large number of persons.

No intoxicated person shall be permitted on a vehicle carrying high explosives. No smoking within 10 ft. of a wagon loaded with high explosives shall be permitted.

(15) All boxes in which high explosives cartridges containing nitroglycerine are packed must be lined with a suitable material that is impervious to liquid nitroglycerine. Cardboard cartons closed at the bottom and made of strong and flexible material that is impervious to nitroglycerine form a satisfactory lining. At least ¼-in. of dry sawdust or wood-pulp or similar material must be spread over the bottom of the box before inserting the cartridge, and all the vacant space in the top must be filled with this material. The cartridges must be so arranged in the boxes that when they are transported with the boxes top side up all cartridges will be on their sides and never on their ends.

(16) Black blasting gunpowder or black and smokeless sporting gunpowder and all military smokeless gunpowder shall be packed in metal kegs containing not more than 25 lbs. Each package containing a high explosive must have the name and brand of the manufacturer, and must be marked conspicuously "Explosive—Dangerous."

(17) No high explosives shall be landed at the piers or elsewhere in the city or transported to a vessel lying at a pier unless the explosives contained in the vessel making delivery are in charge of a duly certified employee of a person, firm or corporation licensed to transport or sell explosives within the city limits. No high explosives shall be landed at the city piers unless for immediate loading into wagons for distribution to consumers for use within 24 hours and for which orders have been previously received, or for immediate transportation by railway to points beyond the city limits. High explosives received at railway stations within the city limits shall be promptly discharged and removed to such storage as these regulations prescribe. High explosives, gunpowder, blasting powder, detonators or electric detonators delivered to railways for transportation to points beyond the city limits must be immediately loaded in a railway car and such car must be forwarded from the loading station within 12 hours from time of delivery. Every car containing high explosives within the city limits must be marked by signs securely attached to the side and the ends of the car, with the words "Explosives—Dangerous" in red letters at least 12 ins. high on a white background.

(18) Every person now engaged with, as principal, or purposing hereafter to be connected with any blasting operations, shall, as hereinafter set forth, make an application to the Chief of the Fire Department for a permit to keep and use high explosives, giving at the time (in writing) the name of licensee, location of office or place of business, occupation, proposed location of the magazine or magazines, together with plans and descriptions and construction of such magazine or magazines, the quantity and kind of high explosives proposed to be kept therein, and the names of employees who should have certificates of fitness.

(19) Magazines for the storage of high explosives within the city shall be of two classes: (A) Magazines of the first class are those which are capable of containing high explosives to the extent of 2,500 lbs. (B) Magazines of the second class are those which are capable of containing not more than 10 kegs of black sporting or blasting powder or 100 lbs. of dynamite.

Magazines of the first class shall be constructed of wood with studding and rafters 2 × 4 ins. or 2 × 6 ins., sheating inside and out, and both sides and roof and places beneath the sheathing filled with dry sand. The outside of the building must be entirely covered with No. 26 or No. 28 galvanized iron to protect it from the water and render it fireproof. Said magazine must be provided with a ventilator at the roof, and magazines of this class shall be lighted from the outside only, and the presence of any form of light or fire within such magazine is hereby strictly prohibited.

Magazines of the second class shall be covered on the outside with iron and on the inside with wood. They shall not exceed 5 ft. in any exterior dimension, nor have an interior capacity of more than 60 cu. ft. This magazine must also be properly ventilated.

Magazines shall contain only the amount of explosives named in the permit, and the placing therein of any other high explosives, percussion caps, detonators or electric detonators, candles, matches, tools, cotton waste, or any article liable to cause an explosion or fire, or any iron, steel or grit is absolutely forbidden.

(20) In the keeping of high explosives from freezing, or in thawing of the same when frozen, only one of two methods shall be permitted. (1) All thawing of frozen dynamite or other high explosive shall be done in a building erected for that purpose, and for which specifications are hereinafter provided. In all places where more than 100 lbs. of dynamite or other high explosive is used daily a thaw house shall be employed of the following construction:

The building shall be divided into two compartments, one for the reception of the dynamite or other high explosive, and one containing the steam or hot water coils. The compartment containing the dynamite of other high explosives shall be supplied with a ventilator at the roof, and the dynamite or other explosives shall be kept in trays or slides which shall be so constructed that same can be removed only from the outside and without entering the building. There shall be no door or other means of ingress entering into said thaw house, and there shall be no door or place to enter between the heating compartment and the compartment containing the dynamite or other high explosive except such opening for the passage of the warm air currents.

For the purpose of heating said thaw house only the use of hot water coils or low pressure steam not to exceed 5 lbs. is permitted. All manner of heat to said thaw house shall be conveyed from the outside of the building by a suitable pipe safely covered. In places where quantities of dynamite or other explosives not to exceed 50 lbs. are used daily, the thawing shall be done in a water-tube thawer, but the water must be heated from outside sources of heat, and the use of a stove, lamp or direct heat in connection with such thawer or process of thawing is hereby prohibited. No blasting caps or primers or detonators or electric detonators shall be kept in any thaw house.

(21) All magazines and thaw houses shall be at all times in the care of a competent employee whose duty it shall be to see that no unauthorized person has access to them. Said employee shall have no other duty that will interfere with his careful supervision of such magazine or thaw house, and he shall have a certificate of fitness as a magazine keeper.

All magazines and thaw houses shall be painted bright red with the words "Magazine—Danger" painted thereon in white letters on a black background. Such letters shall be at least 6 ins. high.

Only persons who hold certificates of fitness, or other authorized persons, shall be permitted to have access to magazines or thaw houses, which shall be kept securely locked when not open for the introduction or removal of high explosives or to inspection by duly authorized officers of the city.

All buildings or places erected to be used as magazines or thaw houses shall first be inspected by the Chief of the Fire Department, and when found to be constructed in compliance with the regulations hereinbefore mentioned a permit shall issue for the use thereof.

No person other than the holder of a certificate of fitness who has duly qualified and whose name has been duly certified by the Chief of the Fire Department to the City Clerk shall in any manner direct any blasting operation or handle high explosives.

(22) In blasting it shall be unlawful to use a quantity not to exceed in disruptive force the equivalent of one pound in weight of 40% dynamite for each 2 ft. depth of hole that is above or less than 10 ft. below the curb; and the equivalent of one pound in weight of 60% dynamite for each 2 ft. depth of hole that is more than 10 ft. below the curb.

Frozen or partly frozen high explosives shall not be placed in drill holes. Frozen cartridges, if not capped, must be returned to the thawing apparatus to be thawed.

The removal of a primer from a frozen cartridge being extremely dangerous is prohibited.

In tamping drill holes wooden rammers only shall be employed. Tamping by strokes is forbidden and only direct application of pressure permitted.

Blasts shall be fixed by the application of some form of electrical current only. Any other method of firing blasts is prohibited.

In order to insure the safety of surrounding property and persons no larger charge shall be used than is necessary to properly start the object it is intended to wreck or blast, and excavating contiguous to any structure shall be so carried on as not to damage such structure.

Weak walls or other supports of such structure must be shored up, and the blasting of decomposed or soft rock is hereby prohibited.

Before firing any blasts, except where the same are in tunnel, the rock to be blasted shall be covered on the top and sides with timber and covered with stout metal matting or some other equally serviceable material to prevent the debris from flying.

The sale, transportation or use of high explosives containing chlorate of potash, perchlorate of potash and picric acid, except as used in blasting caps, is prohibited, except as hereinafter provided.

Any high explosive containing chlorate of potash before same shall be allowed to be sold must conform to the following test: Such mixture must be able to withstand a glancing blow inflicted with a raw hide mallet on soft wood without exploding. Also, such mixture must be so made, as not to reduce the chlorate.

Chlorides calculated as potassium chloride must not exceed 25%.

Any explosive containing chlorate of potash manufactured or offered for sale must contain no free acid, or substance liable to produce free acid.

All explosives containing chlorate of potash or nitrocompounds shall be subject to the British heat test.

The careful handling of all high explosives whether frozen or not frozen, is ordered, and the warming and thawing of high explosives by placing same near fire or against steam pipes is prohibited.

(23) No high explosive cartridge manufactured or sold shall be larger than 4 ins. in diameter and 8 ins. long, and packages of high explosives shall contain not more than 50 lbs.

(24) It shall be unlawful to have more than 1,000 electrical detonators in stock at any time.

The manufacture of high explosives in quantities greater than one pound is prohibited.

The sale, transportation or possession of liquid nitroglycerine except for medical purposes is prohibited.

The sale, transportation or use of high explosives containing chlorate of potash, perchlorate of potash and picric acid, except as used in blasting caps, is prohibited.

The sale, transportation or use of nitroglycerine dynamite containing over 60% of nitroglycerine or gelatine dynamite equal in strength to over 75% of nitroglycerine dynamite is prohibited.

The manufacture, use, sale or transportation of blasting caps containing less than 9½ grains of explosive mixture, at least 80% fulminate of mercury and 20% chlorate of potash, or its equivalent detonating strength, is hereby prohibited.

It shall be unlawful to sell, transport or use leaky dynamite (leaking nitroglycerine).

The keeping or storing of nitro-cellulose in quantities greater than 10 lbs. in one place is hereby prohibited.

The use of frozen or partly frozen cartridges and the breaking or cutting of them or drilling them for caps is hereby prohibited.

(25) If it shall be found that any gunpowder, guncotton, giant powder, dynamite, nitroglycerine, fulminate of mercury, or other high explosives of similar nature, or percussion caps or primers, are being kept in any building, structure, or premises, or in any vehicle or on board of any vessel within the city in violation of any of the provisions of this chapter, any such explosives, etc., shall be immediately seized and removed to such place as the fire marshal may direct. It is hereby made the duty of the members of the police department to assist in making such seizure when requested so to do by the fire marshal, and to assist in the removal of such explosives to such place as may be designated by him.

(26) It shall be unlawful for any person, firm or corporation to manufacture anywhere within the city any gunpowder, guncotton, giant powder, dynamite, nitroglycerine, fulminate of mercury, or other high explosives of similar nature, under a penalty of not less than $50 or more than $200 for each offense. And every day upon which any such explosives are manufactured in the city in violation of any of the provisions of this chapter shall constitute a separate and distinct offense.

(27) Any person, firm or corporation violating any of the provisions of this ordinance, or neglecting or refusing to comply with any of the requirements thereof, shall be fined not less than $25 nor more than $200 for each offense. And each and every day any person, firm or corporation shall violate any of the provisions of this ordinance shall constitute a separate and distinct offense.

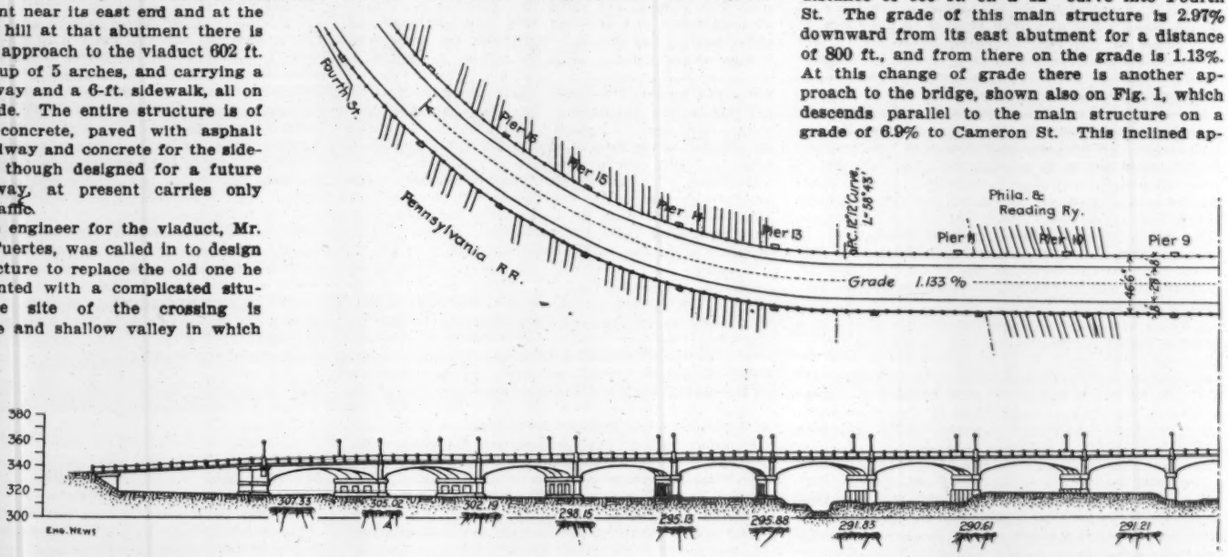
The Mulberry St. Reinforced-Concrete Viaduct, Harrisburg, Pa.

The City of Harrisburg, Pa., has just completed, in its Mulberry St. Viaduct, one of the longest reinforced-concrete bridges in the world. This viaduct proper is 1,841 ft. long, 78 ft. above the ground at its highest point and carries on 19 arches a 28-ft. roadway and two 8-ft. sidewalks. From a point near its east end and at the foot of the hill at that abutment there is an inclined approach to the viaduct 602 ft. long made up of 5 arches, and carrying a 24-ft. roadway and a 6-ft. sidewalk, all on a 6.9% grade. The entire structure is of reinforced concrete, paved with asphalt for the roadway and concrete for the sidewalks, and though designed for a future street railway, at present carries only highway traffic.

When the engineer for the viaduct, Mr. James H. Fuertes, was called in to design a new structure to replace the old one he was confronted with a complicated situation. The site of the crossing is over a wide and shallow valley in which runs a small creek and 27 tracks of the Pennsylvania and Philadelphia & Reading railroads. At this crossing there had been built in 1890-91 a steel bridge, extremely light, and quite insufficient to carry present-day traffic. This bridge entered Mulberry St. practically as an elevated roadway close to the buildings on that street, which itself ran for some distance under the bridge. Some years ago a large fire broke out in these buildings and ruined some of the bridge spans before it could be controlled. This apparent danger from fire and the obvious weakness of the old bridge led the city authorities to appropriate for a new viaduct, but the first appropriation was only $200,000, for a bridge 2,400 ft. long and wide enough for city traffic.

This, then was the problem before the designer: to build for approximately $200,000 a bridge 2,400 ft. long and with at least a 24-ft. roadway, across a network of railway tracks which would hamper all construction work. At one end the center line took a decided curve and a grade of 3% had to be provided, as well as a 600-ft. approach to another street. Finally, the previous fire had made the citizens suspicious of a steel structure and the nearness of the locomotives under the tracks demanded a gas-proof floor system there. Being in the heart of the city, whose citizens of late have developed quite a taste for the "city beautiful," is was also desirable that the question of beauty enter into the design.

To ensure fire-proofness and protection against locomotive gas, concrete was decided upon, and to reduce the cost to a minimum the ribbed system of arch construction was adopted, not only because of the reduction in concrete volume gained thereby, but also because these ribs lend themselves more effectively to a form of erection in which the falsework is the most simple. In solving the various items of the problem here, then, a unique bridge was achieved which not only is admirably suited to its location and needs, but in which the method of erection was both economical and effective.



WEST.

FIG. 1. GENERAL ELEVATION AND PLAN OF
Total length, including approaches, 2,443 ft.

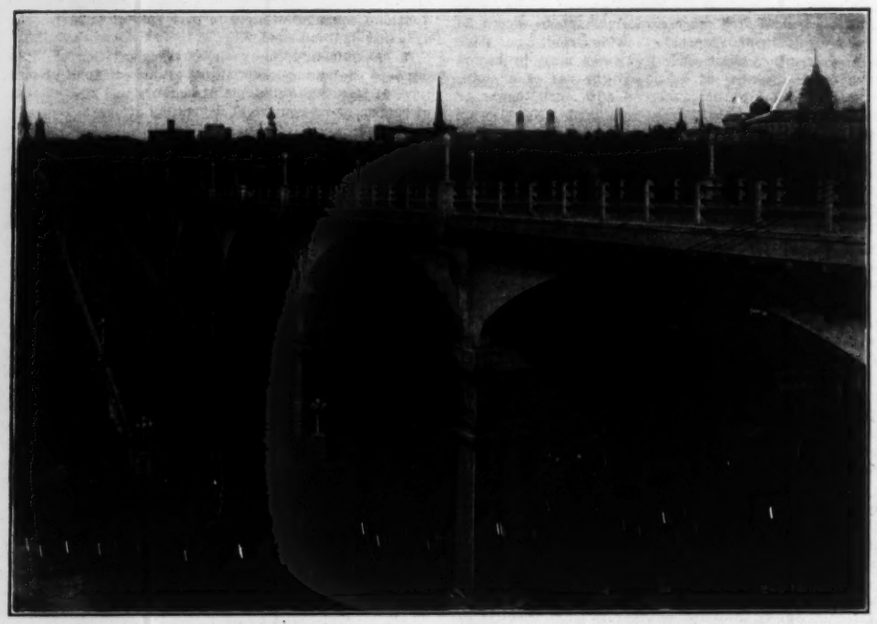


FIG. 2. VIEW OF THE COMPLETED VIADUCT.

Description of Viaduct.

The general layout of the bridge is shown in Fig. 1. The main structure consists of a series of reinforced-concrete arches varying in span from 36 to 93 ft. in the clear, and 40 to 100 ft. c. to c. of piers, carrying a 28-ft. roadway and two 8-ft. sidewalks for a distance of about 1,250 ft. on a tangent extension of Mulberry St. and a distance of 600 ft. on a 12° curve into Fourth St. The grade of this main structure is 2.97% downward from its east abutment for a distance of 800 ft., and from there on the grade is 1.13%. At this change of grade there is another approach to the bridge, shown also on Fig. 1, which descends parallel to the main structure on a grade of 6.9% to Cameron St. This inclined approach has a roadway of 24 ft. and one 6-ft. sidewalk, carried on five reinforced-concrete arches.

Up to the first pier west of the large abutment pier, required by the juncture of the inclined approach and the main viaduct, all the piers are normal to the center line of the bridge and of the inclined approach, but west of that pier the direction of the railway tracks governs the pier positions, so that length and skew of the arches is extremely variable.

Starting from the east abutment the arches have the spans, c. to c. of piers, noted below (the spans of the inclined approach are of the same length as the adjoining spans on the main bridge):

1— 61.5		
2—100.0		
3—100.0	9— 40.2	15— 77.0
4— 94.0	10— 77.0	16— 74.0
5—100.0	11— 71.6, 75.1, 78.9, 82.4	17— 83.0, 82.3, 81.6, 81.0
6—100.0	12— 85.0	18— 75.0, 75.3, 75.6, 76.0
7— 100.0	13— 83.0	
8— 100.0	14— 76.6, 72.8, 68.4, 64.4	19 86.8, 76.6, 65.3, 54.8

The 11th, 14th, 17th, 18th and 19th arches are on curves and skews so that each of the four ribs are of different length.

Arch Details.

Although the various arches differ in length of span and degree of skew they are all of the same general type. In Fig. 3 are shown the details of one of the 100-ft. right arches, which is typical enough of the general design to cover the details of the other spans. From the half-tones and from the general layout in Fig. 1 it will be seen that the showing face of the spans is a flat three-centered curve, spanning between high slender piers. Structurally the span is composed of four arch ribs carrying transverse floor-beams and a reinforced floor slab. These ribs are spaced as shown in Fig. 3 and vary from 22 to 26 ins. in thickness depending on the span and their position under the floor. In appearance they resemble beams with a reduced middle section, but they are reinforced and designed as arches, as shown in some of the typical spans in Fig. 3. The con-

crete above the extrados reinforcement was not considered in the computations for rib strength.

All rib reinforcement is of 1-in. section rods arranged according to the demands of loading. The horizontal rods at the top of the beams were placed largely for erection stresses, and the general design of the arch ribs is a result of the method of erection. That scheme of erection is as follows:

It will be seen that the piers between the arches are very slender and not designed to take the thrust from more than the two arches which they each support. In order to avoid building these piers heavier or even constructing occasional abutment piers a scheme of erection was worked out by means of which the arches became temporarily cantilevers. In first studying this question it was thought best to use centering which could be attached to the pier at the base of the arch rib and cantilevered out day by day as the work progressed, but the difficulties of bracing such a system of centers were so great that it was decided to build steel forms spanning from pier to pier with no supports between. With these forms, which are described fully later in this article, the cantilever method of construction was applied to the concrete alone.

Briefly this consisted in placing across the top of the arch ribs a continuous line of horizontal reinforcement which, after the arches had met at the center, was cut over each pier. Thus, as concrete laying was started at each pier and continued symmetrically on either side until the adjoining crowns were met, each pier was a unit, carrying on either side as cantilevers, the two arch ribs. No stress from adjacent arches was therefore carried to any pier. Fig. 3 shows how this construction was carried on. First, the pier was completed to its coping, where a sloped surface was left for a skewback on either side. The steel form was then placed and the reinforcement arranged in position both in the arch rib and along the horizontal plane at the top of the pier. Concrete was then laid on one side of the middle line of the pier for a distance of about 10 ft. from the center line, sloping up from the intrados to the top of the pier in a line normal to the neutral axis of the arch. The face of this concrete on this middle line of the pier was in the lower part placed against a sheet of corrugated iron, and at the upper part left back from the middle line so as to provide an opening in which the horizontal reinforcement might afterward be cut in two. After this piece had hardened the corrugated iron was removed and a similar section on the other side placed in a like manner. The next day the section adjacent to that first laid was deposited and so on, continuing the arch in a balanced loading. In this way the wet concrete came directly on the forms of the centering, but upon hardening the greater part of its load was carried as a cantilever to the pier, through the tension in the horizontal rods. The corrugated iron left a rough vertical surface which served to transmit the shear from one overhang to the other when they were unbalanced.

This process was continued successively from one abutment to the other and when the final crown connection had been made and the proper set reached, the horizontal rods were sawed apart over each pier, leaving the opening there to be spanned with the floor.

For transverse bracing the outside arch ribs are connected along the haunches by a slab of reinforced concrete (Fig. 3) and over the middle of the piers by a vertical diaphragm (Fig. 4). The two middle ribs are not joined together.

The floor is a simple flat reinforced-concrete 8-in. slab, resting on 11 x 9-in. floor-beams, spanning and resting directly on the arch ribs. On this floor, separated by a thin fill, is an asphalt roadway and concrete sidewalk. Provision has been made for a future street railway, but at present there is none on the viaduct.

The details for the inclined approach are practically identical with those on the main bridge, except that only three arch ribs are used instead of four.

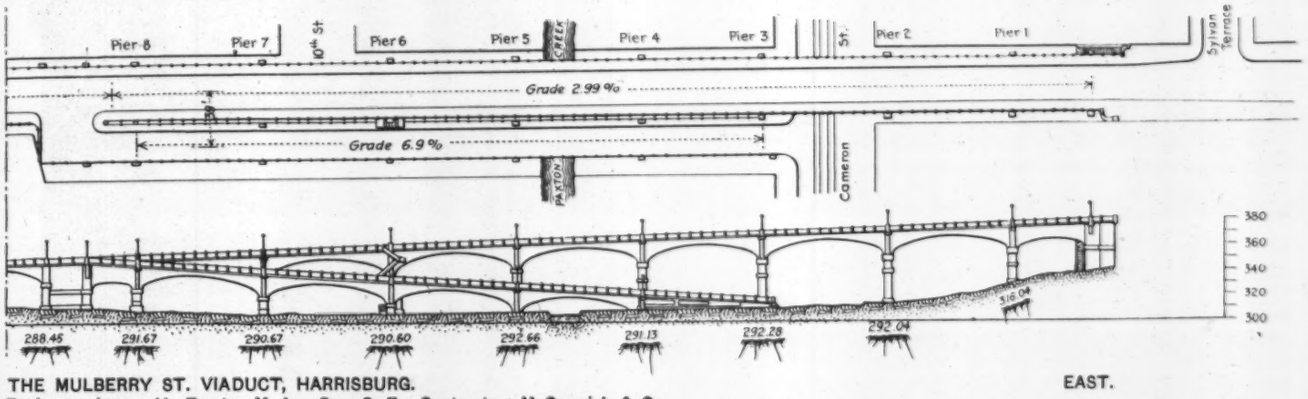
EXPANSION.—As explained above the arches were made independent of each other after the centers were removed so that they were free to move in their upper planes. On this account the effect of temperature in changing the length of the bridge is at a minimum. However, to provide for expansion and construction of the floor system, a sliding joint is made in each floor slab at the top of the pier alongside of the opening left in which to cut the horizontal rods (Fig. 3).

Piers and Abutments.

The piers are of several styles, according to their position and needs, but they all follow the same general type above ground. Fig. 5 shows a vista through the piers on the level and line of Mulberry St. looking west from Cameron St. and illustrates very well the main type of pier. This, as shown in detail in Fig. 4, consists of a solid plinth immediately under the skewbacks, carried on two slender reinforced piers to solid rock. Where the clearance was sufficient, as in Fig. 5, the connection between the two columns is made with an arch, but where the elevation of the top of the pier was low, as in Fig. 6, this connection was made with a heavily reinforced-concrete beam. In this latter case the decoration of the pier was much simpler.

At the upper end of the Cameron-St. approach the design required an abutment pier, not only on account of the unbalanced loads from uneven spans on either side of the pier, but because of the two arches coming into one side as against one arch on the other. This pier then, is of unique design. It is shown in Figs. 7-8. If at this point a solid pier had been used, the unbalanced load would have required a base much longer and a pier much heavier than economy or appearances would allow. To avoid this heavy pier a short arch was interposed between the last pier of the 100-ft. arches and the real abutment pier. This small arch, of 40-ft. span, is heavily reinforced as a beam and serves to lighten up the construction and to transmit the stresses over into the main pier.

This pier (Fig. 7) is of the cellular type, with



THE MULBERRY ST. VIADUCT, HARRISBURG. EAST.
Engineer; James H. Fuertes, M. Am. Soc. C. E. Contractor: McCormick & Co.

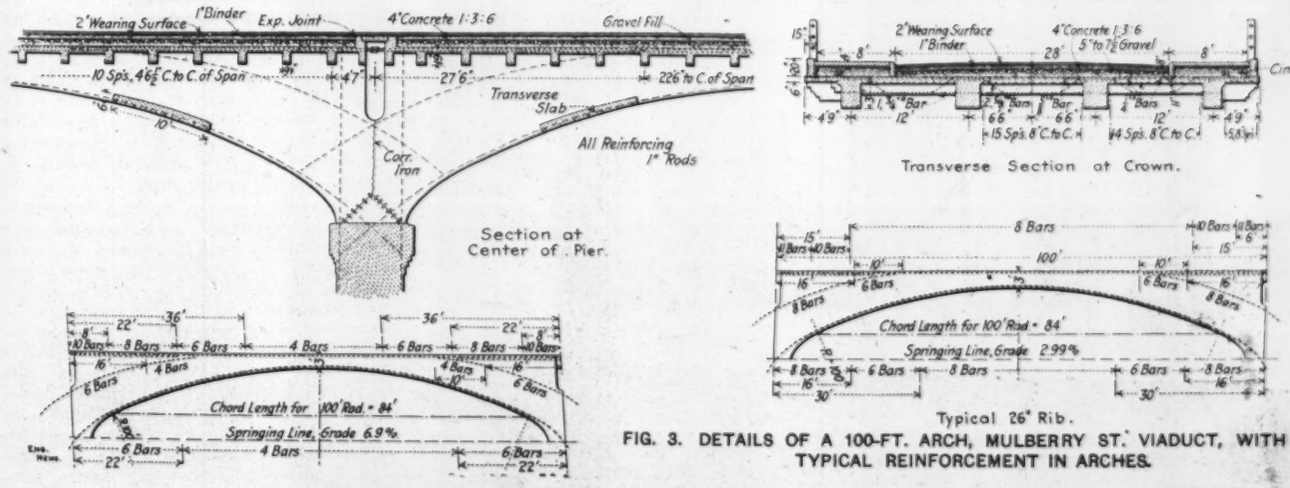


FIG. 3. DETAILS OF A 100-FT. ARCH, MULBERRY ST. VIADUCT, WITH TYPICAL REINFORCEMENT IN ARCHES.

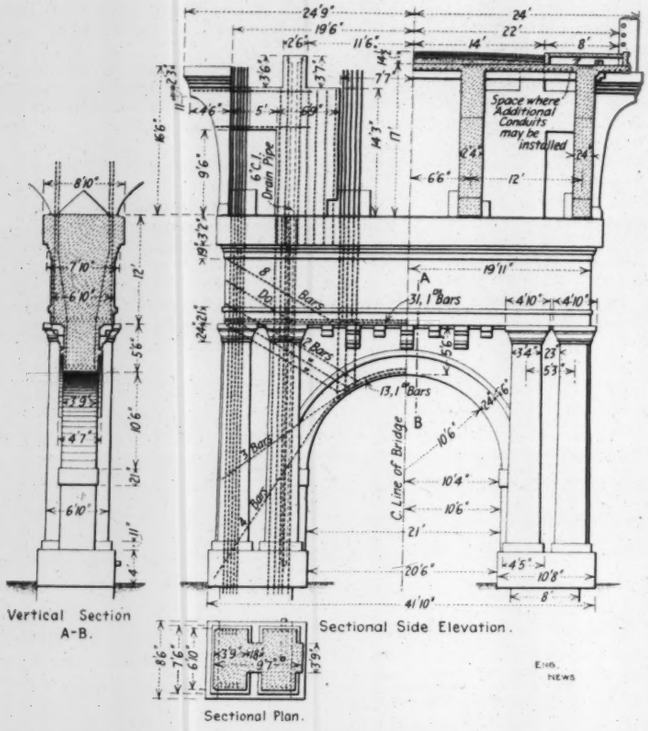


FIG. 4. DETAILS OF A PIER FOR THE 100-FT. SPANS.

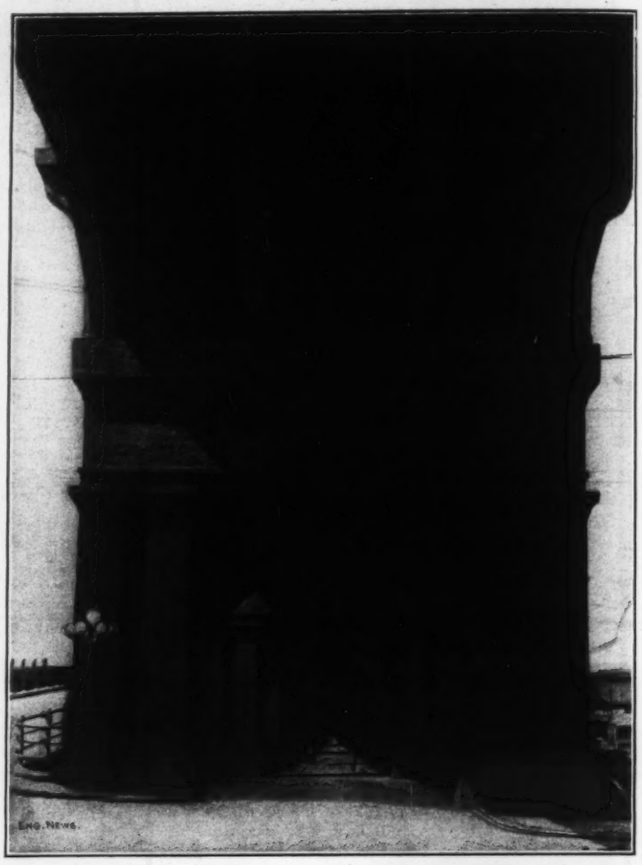


FIG. 5. VISTA UNDER PIERS, LOOKING FROM FOOT OF CAMERON ST. APPROACH.

each of the four main arches on either side and the three approaches on one side, continued down as the longitudinal walls of the cells. These walls increase in thickness as they descend and are reinforced as arch abutments to the bottom. They are connected at the middle and on the ends by similar walls running transversely.

The abutments, of which there are three, are of the U-shaped type with stepped face and side walls and buttresses running back in line of the arch ribs to solid rock.

At some of the piers the footings of the old bridge were utilized for the new by enlarging their sections with concrete.

Ornaments and Decoration.

Because of the location of the viaduct in the middle of the city, an especial effort has been made to give it a pleasing appearance. To that end considerable care has been taken with the architectural design, as the views here given of the finished structure will show. No little part of this artistic merit is due to the admirable surface which has been bush hammered on all showing faces.

At numerous places along the viaduct stairways have been built around the piers giving access from the ground to the roadway. These are built of reinforced concrete on cantilevers out from the pier. The railing all along the viaduct is of concrete blocks cast in the yard and connected with pipe, placed in sleeves which will permit of movement if expansion takes place. At every pier and sometimes in between is placed a lamppost of cast concrete, carrying a spherical globe and at the main piers and abutments, a cluster of globes. These posts are exceptionally well executed and are of tasteful design. This type was cast in a shop, the vertical shaft and each of the brackets as a separate piece and put together in place.

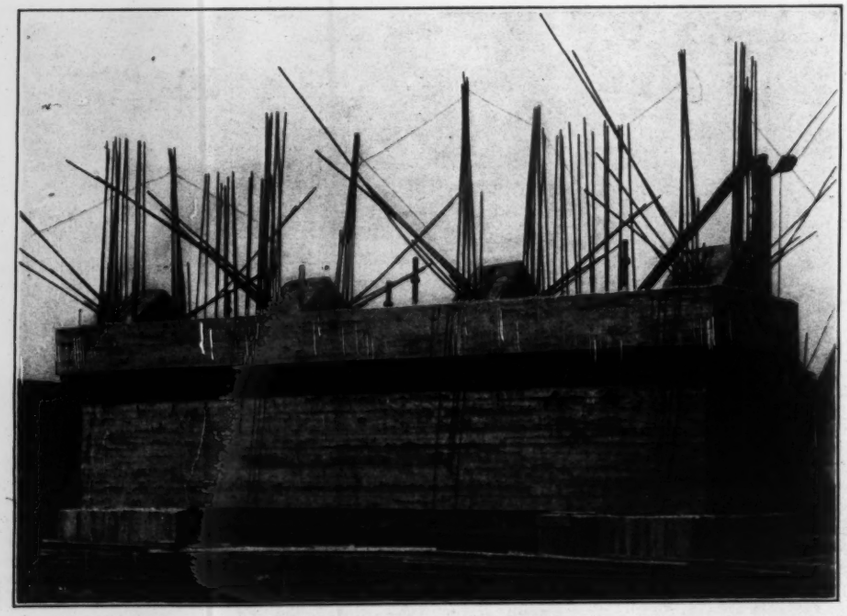


FIG. 6. ONE OF THE LOWER PIERS.

Steel Centering.

Briefly the centering consisted of a steel box plate-girder spanning from pier to pier at each rib, but the peculiar method of erection, noted above, and the great variation in the length of the spans to all of which the forms had to be adjustable, brought about quite a complicated design. As shown in Figs. 9-12 the centering for each rib was a box girder in three parts, two with sloping bottom from each end to the middle, and a middle section of constant depth. Each of these three sections was complete in itself, but they were all three bolted together at their ends when in place. It will be noted that the middle section has vertical stiffeners at irregular spacing. These stiffeners were so arranged that by cutting off the girder at a stiffener, the shorter piece would be just the correct length to use, with the two side pieces as they were, for the centering of one of the shorter spans. Thus, the longer spans were first built and the form unbolted at the section joints, the middle piece cut to the correct length and the form used for the next shorter span. The upper edge of the form in all spans came just below the sidewalk corbels, and the upper part of the spandrel wall and the corbels were laid inside of timber forms fastened to the top of the steel box-girders. The intrados of the arch, which, of course, varied in each of the length spans, was formed against lagging held inside of the box girder by bolts through the stiffeners, and the bolt-holes were so punched in these

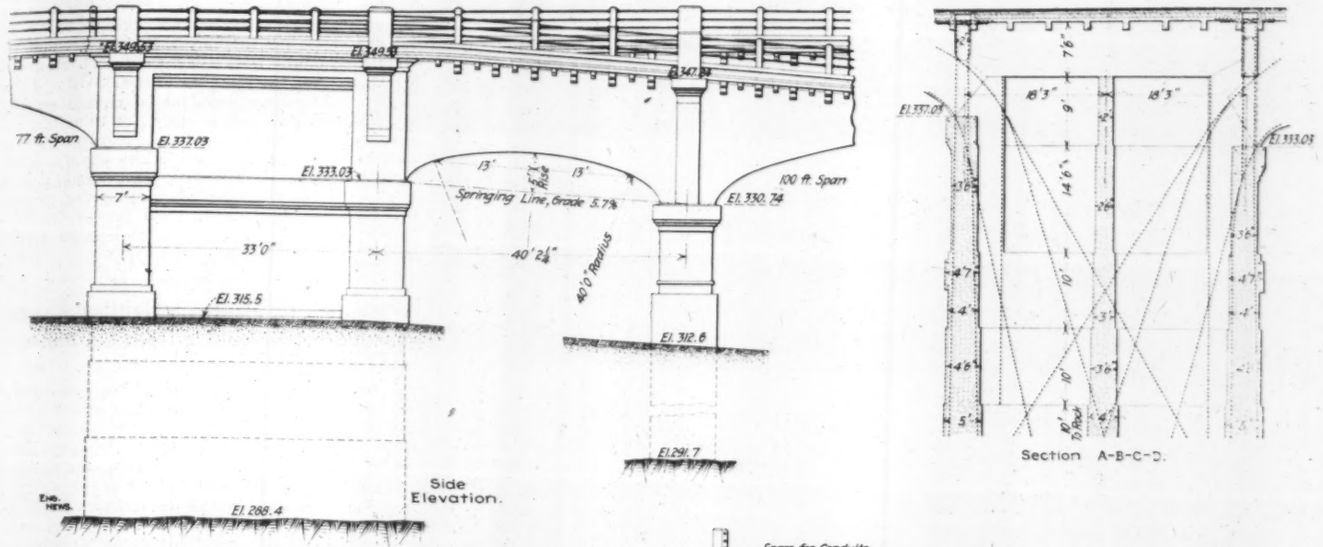
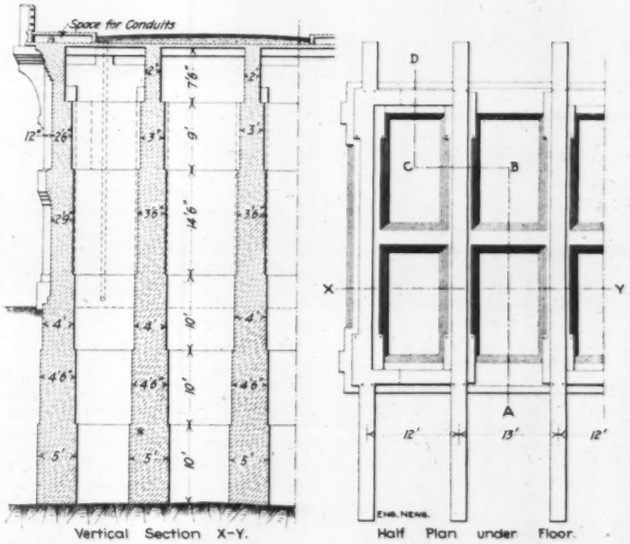


FIG. 7. DETAILS OF MIDDLE ABUTMENT PIER, MULBERRY ST. VIADUCT, HARRISBURG, PA.



stiffeners that the position of the lagging could be regulated to fit the curve of the intrados.

Each box girder was tied together at top and bottom by occasional tie-plates and the four forms for each arch thoroughly interbraced with timber braces and steel tie-rods. The inside face of the forms was smooth, all rivets being countersunk, so that the concrete surface left was more than unusually clean and smooth.

The bearing at the pier is shown in Fig. 11. First, the pier is completed to the level of the coping except at the line of each arch rib, the skewback was built up on each side to a normal to the rib. Then on the inside of each of these skewbacks two 20-in. I-beams were laid on the flat top of the arch, in line of the bridge and projecting over each side of the pier. Through each end of each of these two beams, which were bolted down to the top of the pier with two 5-in. bolts each, and which were tied together by bolts and separators, hanger rods were suspended from the end of which another double row of I-beams, 20 ins. deep, was hung parallel to each transverse face of the pier and immediately alongside of it. On these transverse beams at each skewback and on each side there were laid three 8 × 8 × 16-in. pine blocks, on top of which a steel plate carried the base of the box-girder forms. This base consisted of two steel angles bolted together to form a T, the stem of which acted as a knife edge support, and the horizontal face of which was riveted onto the bottom face of the girder. (See Fig. 9.)

In order to transmit the load of the cantilevered concrete in one form back into the adjacent form and so balance the load over each pier, ties were provided across the top of the piers between adjoining girders. These ties can be seen in Fig. 10. They consisted of 2¾-in. rods threaded on the ends and provided with nuts and washers which backed against two channels, filled between with a wood block, and laid across the top angles of each box girder. The channels in turn backed against short lengths of old rails riveted longitudinally to those same top-angles. As soon as the forms were swung into place (see Fig. 12) these tie-rods were tightened up to give the proper camber to the girders and were kept tight all through the laying of the concrete. The concrete finally surrounded the I-beams, but the nuts, being outside of the girders, could be unloosened and the girders let away from the rods, which were left tying the adjoining arches together until needed for the erection of other forms in the next span, when the nuts were removed and the rods taken out.

In the lower part of the girder forms, just above the angle knife-edge support, there was placed a stepped set of angles, as shown in Fig. 9. In laying the arch rib the concrete was first laid on the skewback and up to the box girder past these small steps. It was then allowed to set, and in all future concreting the in and up kick of the foot of the form against the pier, a kick due to the reaction from the cantilever loading on the arch, was taken care of partly by these steps of concrete projecting from the side of the pier. After the forms were taken down these steps were removed and the intrados made smooth by tooling. On some piers the forms for each rib were bolted together on the ground and swung into place by locomotive cranes on the tracks below (Fig. 12), but in the high spans the

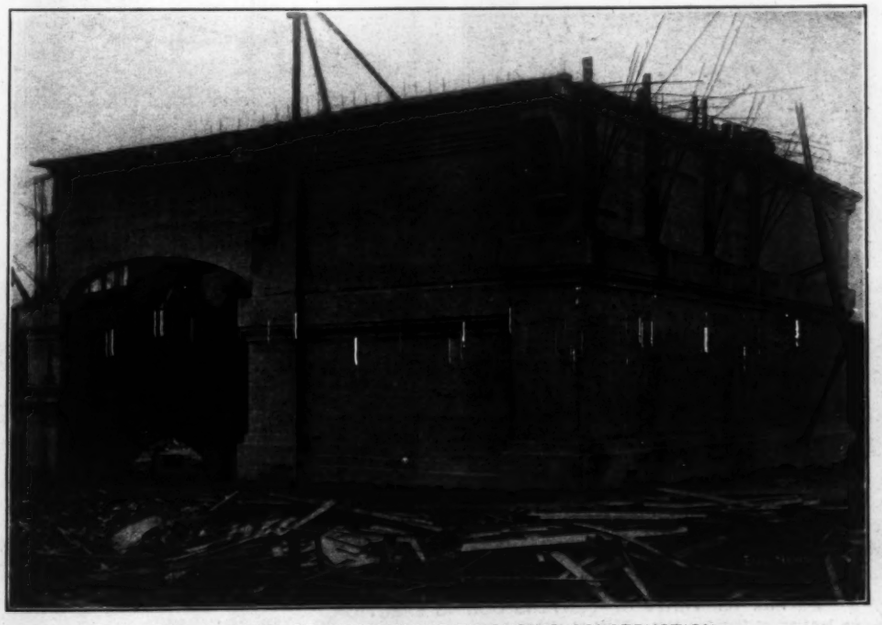


FIG. 8. VIEW OF ABUTMENT PIER DURING CONSTRUCTION.

forms were erected in three pieces, by the aid of derricks, and bolted together, while hanging from these derricks.

Of these box-girder forms 24 were provided, so that six complete arch spans could be in the course of construction at one time. On account of the difficulty in arranging the stiffeners so as to be cut to the right length, three extra middle sections had to be fabricated. With the exception of two short spans, all of the main and approach arches were laid in the forms. As on the curve no two of the ribs, in one span even, had the same length the adaptability of this style of form to any length made it very valuable.

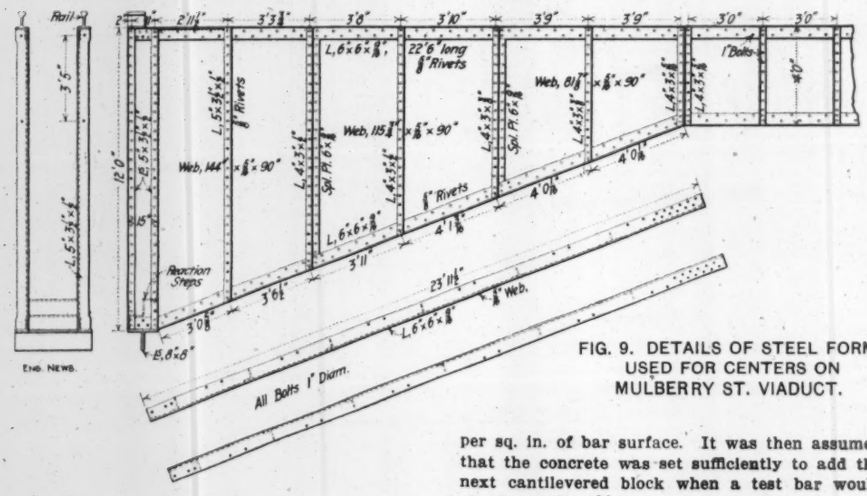


FIG. 9. DETAILS OF STEEL FORMS USED FOR CENTERS ON MULBERRY ST. VIADUCT.

Material.

The concrete used was of different classes, a 1:3:6 mixture for the foundations, a 1:3:5 for piers and retaining walls and 1:2:4 for their reinforced members, such as floor slabs or arch ribs. To the piers and footings, stones up to one man size were allowed, if due regard were given to spacing. Reinforcing rods were generally of twisted 1-in. square bars of steel, the number being varied to give any desired section.

For plain concrete in direct compression a unit-pressure of 300 lbs. per sq. in. was allowed, but for compression in concrete reinforced the unit stress was increased to 500 lbs. per sq. in. The design was made on an assumed load of 200 lbs. per sq. ft. over the entire roadway or for a concentrated load of a heavy street car; and for a load of 150 lbs. per sq. ft. on sidewalk.

Fig. 13 shows a very interesting diagram of concrete laying on the seven 100-ft. arches, from Pier 1 to 8. This diagram gives the time taken to erect the steel centers, to construct the wooden forms on top of these centers, to lay the concrete, and to take down the forms and centers, all recorded by daily totals. From this it will be seen that in the warm fall concrete laying proceeded at a fairly regular rate, but that as cold weather set in, about the middle of November, the interval between the laying of successive sections noticeably increased. This was due to the greater length of time that it took the concrete to set as the thermometer went lower.

Incidentally, a novel method of determining the strength of the concrete was used by Mr. Fuertes. It was computed that the maximum increment of stress to which the horizontal rods at the top of the pier would be subjected would be 18 lbs. per sq. in. of bar surface. It was then assumed that the concrete was set sufficiently to add the next cantilevered block when a test bar would stand a pull of 80 lbs. per sq. in. This, however, was afterward reduced to 40 lbs. The test was made by inserting in each batch of newly laid concrete ⅝-in. twisted steel rods with loops on their upper end and then by leverage determining the load sufficient to just budge the rod. When this leverage reached the required 40 lbs. per sq. in. of embedment the concrete was considered strong enough to proceed with the work.

Under this scheme, as winter came on, the period of setting was observed to be lengthening and consequently the work was being held back. On this account, after about the last part of November, the concrete in the forms was kept heated through the setting period, with the result that the period of setting was shortened and the curve of depositing regained its original shape.

The viaduct was built by the Board of Public Works of the City of Harrisburg, then consisting of Messrs. Wm. Jennings, David E. Tracy, A. C. Stamm and Edwin C. Thompson. Mr. James H. Fuertes, M. Am. Soc. C. E., was the engineer in charge of design and construction, and to him is due the credit for the successful and economical completion of the work. Mr. George H. Maurice, Principal Assistant Engineer of the Board of Public Works, was the engineer in charge on the site. Messrs. McCormick & Co., of Philadelphia, were the contractors. As the main bridge is 1,841 ft. long and 44 ft. wide and the approach 602 ft. long and 30 ft. wide, a total roadway area of about 100,000 sq. ft., the cost of the bridge, $265,000, was only about $2.60 per sq. ft.; remarkably low for this style of permanent structure.

FIG. 10. VIEWS OF CENTERS DURING ERECTION.
(View at left shows bearing of form on pier. View at right shows the timber forms at top of brackets and coping, and at left the connecting rod between two forms.)

Locomotives vs. Stationary Hoisting Engines for Raising Material in Open Cut Mines.

By J. F. JACKSON,* Assoc. M. Am. Soc. C. E.

A marvel of the mining world for some years has been the wonderful open pit mining as conducted on the Missabe iron range in northern Minnesota, and lately in certain porphyry properties in the west producing large tonnages of copper ores. Overburden in the shape of rock or earth from 20 ft. to 50 ft. or more in depth is stripped from areas of ore beds often hundreds of acres in extent by steam shovel and train, or other methods. The horizontal ore beds thus exposed are often 100 ft. to 300 ft. or more in thickness. The nature of these iron and copper ores made it possible to load them with steam shovels direct into ore cars with comparatively little blasting.

The demands of this work have developed the steam shovel to the dependable machine of today which has made the Panama Canal and many other great works possible. In the early stages of this wonderfully economical steam shovel mining, the obvious thing to do was to handle ore trains direct into the mining pits by means of locomotives. Many interesting and complicated systems of inclined grades, curves and spirals have been worked out, and as the mines have increased in depth it has required constant ingenuity on the part of mining engineers and operators to keep the railway grades

*Contracting Engineer, Wisconsin Bridge & Iron Works, Houghton, Mich., and Milwaukee, Wis.

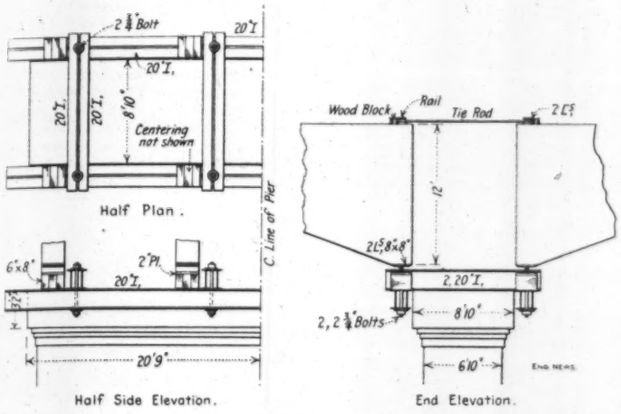


FIG. 11. DETAILS OF BEARING OF STEEL FORMS ON PIER.

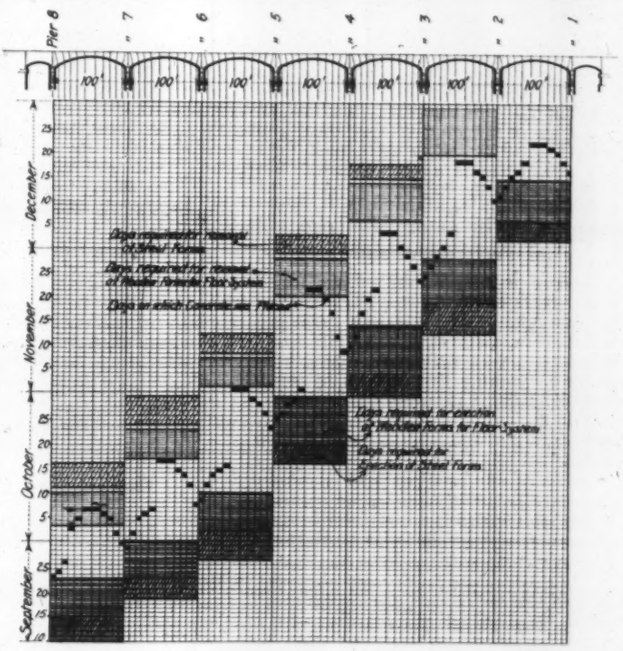


FIG. 13. DIAGRAM OF CONCRETE LAYING AND FORM CONSTRUCTION ON MULBERRY ST. VIADUCT.

under consideration the problem of whether to build a half-mile trestle on a 2% grade to gain access to the top of stamp mills at an elevation of about 60 ft. above the general level of the railway, or to use steel inclines with hoisting apparatus. It was decided to use inclines, and one mill was equipped with a 20% incline up which a single ore car of 40 tons capacity was hauled by a simple second-motion Nordberg hoisting engine. The other mill, near and beyond the first, was equipped with an 11% incline, up which two 40-ton cars are hauled at one time. These inclines have been in operation several years without accident, handling up to 4,000 tons per day, at a very low cost per ton. Without going into detailed figures at this time, I wish to assert that the capacity of a single-track incline on a 10% or 12% grade is very much greater than the capacity of a single-track railway line on any grade reaching to the same vertical depth into the mine, when that depth exceeds say 100 ft. or perhaps less.

To put the matter in a striking manner, take the operation of an underground mine having a vertical or steeply inclined shaft. It would hardly be proposed seriously to lower the winding engine into the shaft each time it was desired to hoist out a load, and make the engine hoist itself out with the load every time. Assume now a covered or open shaft or incline on an 8% grade, or having a dip of 7%. Suppose further that an ordinary first or second motion hoist (such as is used around mines) weighing say 30 or 40 tons is mounted on a flat car, the car loaded with fuel and water, and the hoisting rope fastened to an anchorage above the collar of the shaft. You are now ready to lower the car and hoist into the mine, couple onto two, three or four ore cars, and hoist the movable hoisting plant with its train of ore out of the mine.

All this sounds somewhat preposterous, but mechanically it is precisely what is done when ore is hoisted out of a mine on excessively heavy grades with a locomotive. A locomotive is capable of exerting a drawbar pull or tractive effort of 20% (or a little more) of that portion of its weight which rests on the drivers, due to the limits of coefficient of friction of tires on rails.

A certain ten-wheel (4-6-0) engine with tender weighs 140 tons, of which 70 tons is on the drivers. At 20% friction, it is just able to pull itself up a 10% grade. On a 5% grade it can pull itself and a load equal to itself. By the use of sand a slightly better friction coefficient can be obtained, against which a tractive or rolling resistance of 8 or 10 lbs. per ton for slow moving must be figured. Shay (geared) and saddle-tank locomotives have the advantage of a greater portion of load on drivers. For the sake of simplicity, assume 20% friction, and neglect tractive resistance, curve resistance, etc.

For the engine above mentioned you can haul the following on a 5% grade: (1) the locomotive, 140 tons; (2) two cars, 40 tons; (3) two loads of ore, 100 tons; total train, 280 tons. Assume a 72-ton Shay (geared) locomotive with all the weight on drivers; and a grade of 8%. Total drawbar pull at 20%, 28,800. The load (12 times 28,800), say 172 tons. Of this total, the engine weighs 72 tons; the two cars, 30 tons; and the two loads of ore, 70 tons.

The above figures may be bettered slightly, but they illustrate without any complications the principle involved; namely, that it is folly to make your hoisting engine expend a great part of its energy in hoisting itself.

No one believes that any method more economical can be found than stoping by steam shovel and tramming by locomotive and 50-ton cars. Can we then retain the advantages of steam shovel stoping and locomotive tramming, without the disadvantage of locomotive hoisting? We can do this even more advantageously than at present, and without sacrificing tonnage, by simply planting a heavy-duty hoisting engine at the top of an incline having a grade of 9% to 10% which extends to the deepest part of the mine, with turnouts to each higher level. These levels may be 15 ft. to 25 ft. apart vertically, to suit the convenience of the stoping.

We can hoist two or three loaded cars at a time, and lower a corresponding number of empty cars each time, doing practically all of the switching at top and bottom by gravity, as in the case referred to above. There is no feature of the manipulation of these cars at top or bottom of the incline which has not been successfully worked out and operated for years, as may easily be shown. The resulting economy and convenience in every phase of the work could not fail to be great. Levels would be kept nearly level, sloping slightly toward the "shaft" at each level. Tramming would be by light locomotives handling but few cars at a time.

A hoisting engine with cable having a safe working strength of 25,000 lbs. will handle two 40-ton ore cars (weighing 60 tons each when loaded) up a 10% grade with ease and great dispatch. Such a grade 1,000 ft. long will reach to a depth of 100 ft. When the mine is deep, grades up to 20%, on which to handle single standard ore cars, may be used and attain the desired depth within a shorter distance. The necessary amount of switching at the shaft (all of which should be done by gravity) is slightly

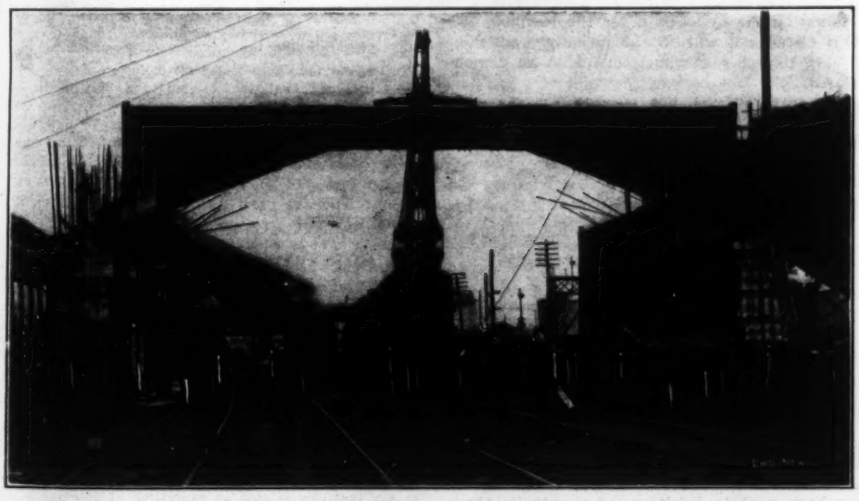


FIG. 12. LIFTING ONE OF THE STEEL FORMS INTO PLACE WITH A LOCOMOTIVE DERRICK.

increased; otherwise the economy and daily capacity are but little affected.

In conclusion let me say that first cost, cost of operation and maintenance of a stationary winding engine will be much less than that of an auto-tractive hoisting plant, and that the continued use of the locomotive on heavy grades for mine hoisting is simply an example of a good idea carried beyond its proper limits.

Coal Hauling by Automobile Truck: Record of Performance and Costs.*

A record of the performance of a 10-ton automobile truck is given in the accompanying table. This data was recorded during a half-month of actual operation in New York City hauling coal from a coal yard to various points of delivery. In the 13 working days covered by the table, a total of nearly 1,100 tons was delivered at an average distance of 2.13 miles from the base with an average fuel consumption of 12 gals. of gasoline per day.

The truck was equipped with a four-cylinder, four-stroke-cycle gasoline engine developing 36 B. HP. at 1,000 r. p. m., planetary transmission (two speeds and reverse) and double-chain drive. A special chute was put in at the coal yard by means of which the truck could be loaded in 1½ minutes. This arrangement greatly helped to increase the number of loads per day. The car showed no abnormal wear and there were no breakages nor any serious deterioration of parts. At the time the record was taken, the car had been in regular use nearly four months. At the present writing, a little more than a month after the final date of the trial, the tires are reported to be still in excellent condition and it is estimated that they will complete a year of service. This indicates that the rate of operation shown in the table and maintained in the regular use of this particular truck was not excessive or in any sense an overload.

The distances shown in the final column of the table are the total distances traveled in delivering the corresponding number of loads indicated in the second column, including return trips to the coal yard. These distances were measured on the city map. The third column gives the number of gallons of gasoline put into the 27-gal. tank each day, but this has no relation to the daily consumption of gasoline, of which no record was kept. The total consumption during the period covered by the table was found by adding together the various amounts put into the tank, taking care that it was filled to the same depth at the beginning and the end of that period.

The cost per day ($16) is estimated on a basis of five years' operation. It represents what would be the average cost per day if the truck were operated for five consecutive years and then discarded, and it includes a 20% depreciation charge, interest at 5%, labor, tires, yearly overhaul and current repairs, gasoline, lubricating oil, insurance and garage charges.

*Information furnished by the Hewitt Motor Co., 6 East 31st St., New York City.

RECORD OF PERFORMANCE OF TEN-TON GASOLINE MOTOR TRUCK DELIVERING COAL.*

Date	Loads.	Gasoline, gals.	Day's Mileage.	Total Distance.
Oct. 13	1	12	37.0	5.5
"	2	..	....	10.0
"	1	..	....	3.5
"	4	..	....	18.0
14	1	8	37.0	5.5
"	7	..	....	31.5
15	1	10	38.0	5.5
"	1	..	....	5.5
"	6	..	....	27.0
16	5	12	34.0	22.5
"	1	..	....	5.5
"	1	..	....	3.0
"	1	..	....	3.0
18	1	12	36.5	5.5
"	1	..	....	4.0
"	6	..	....	27.0
19	2	15	36.5	7.0
"	3	..	....	13.5
"	4	..	....	16.0
20	1	11	33.5	5.5
"	3	..	....	10.5
"	1	..	....	4.0
"	3	..	....	13.5
21	8	10	34.6	32.0
"	1	..	....	2.6
22	9	12	40.0	40.0
23	8	17	33.5	28.0
"	1	..	....	5.5
25	1	10	37.0	5.5
"	7	..	....	31.5
26	3	14	37.8	13.5
"	3	..	....	13.8
"	1	..	....	5.5
"	1	..	....	5.0
27	1	12	34.3	5.0
"	1	..	....	2.0
"	3	..	....	13.8
"	3	..	....	13.5

Average miles per day	35.4
Average gallons gasoline per day	11.9
Average miles per gallon	2.97
Average number of loads per day	8.3
Average tons per day	84.03
Total tons in 13 days	1,092.39
Total pounds in 13 days	2,184,640
Cost per day (maximum)	$16.00
Cost per ton	$0.19
Average miles from base	2.13
Cost per ton-mile	$0.09
Weight of car, empty, lbs.	13,000
Average weight of load, lbs.	20,250
Total weight, lbs.	33,250
Average rolling load, lbs.	23,125
Average ton-miles per gallon of gasoline	34.43

*Truck owned and operated by Burns Bros., New York City; made by Hewitt Motor Co., 6 East 31st St., New York City.

Vitrified Clay Curbing for Streets and Roads.

A recent development in street and road paving is the use of vitrified blocks for curbing, and as the blocks are hollow they form continuous drains, so that the usual line of broken stone for the drainage of the roadbed is not required where this form of curb is used. The blocks, shown in section in Fig. 1, are 24 ins. long and weigh about

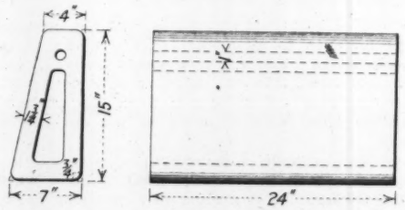


Fig. 1. Vitrified Clay Block for Street and Road Curbing.

130 lbs. each (65 lbs. per ft.). Tests have shown the safe crushing strength to be about 10,000 lbs. per sq. in. The curbing can be laid at about the same cost as concrete, but is said to have the advantage of being more easily laid and by ordinary labor. It is superior to brick owing to its greater size and stability, and is considerably cheaper than stone. It effects a saving also in eliminating the underdrain for the roadbed. To what distance it will be economically practicable to ship this clay curbing remains to be demonstrated.

This form of curb has been in use for eight years on the country road between Toledo and Calumet, O., and is said to be in excellent condition, showing practically no wear. It is now being introduced more generally, however, and has been used in brick paved streets in Akron and Empire, O. It is to be laid also on the Woodland Road leading out of Cleveland, O., and about 30 miles have been ordered for use in Cuyahoga County, Ohio, during next year. Fig. 2 shows the curbing applied to a 30-ft. street and to a 15-ft. highway, both with brick paving. The curbing is manufactured by the American Sewer Pipe Co., of Akron, O., which has furnished us with information concerning it.

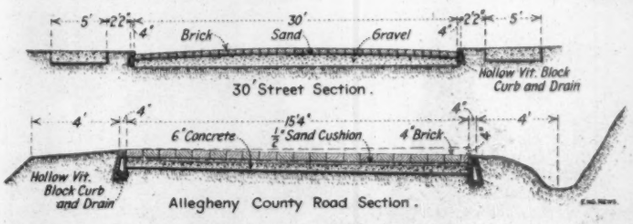


FIG. 2. BRICK-PAVED STREET AND HIGHWAY WITH VITRIFIED CLAY CURBING.

FULTON MEMORIAL WATER GATE.—The jury of award of the Robert Fulton Monument Association announced, on Jan. 6, the ten successful entrants in the preliminary architectural competition for the design of a permanent water gate on the Hudson River at Riverside Drive near One Hundred and Tenth St., New York City. This structure would replace the temporary pavillion, erected at the time of the Hudson-Fulton celebration, for the reception of naval guests, etc. The present intention is to construct an imposing landing place, at an expense of about $2,500,000. The money is to be raised by subscriptions gathered in all parts of the country. It is planned to make this gateway a memorial to Robert Fulton solely, no consideration being given, apparently, to commemorating the works of the earlier inventors who assisted in the development of the steamboat.

There were 62 designs submitted. The jury of award consisted of two architects, Thomas Hastings and George B. Post, and two laymen, Robert Fulton Cutting and Isaac Guggenheim, with Lansing C. Holden as advisory architect. Each of the ten successful competitors received a prize of $500. The successful contestants were Charles P. Huntington, Mills & Greenleaf, Lawrence F. Peck, J. H. Freedlander, Bosworth & Holden, and Harold Van Buren Magonigle, of New York City; Robert P. Bellows, of Boston; Albert Kelsey, Albert C. Cret, and Heacock & Hokanson, of Philadelphia, and Herbert Scott Olin, of Watertown, N. Y.

These architects will be required to submit final designs on March 1, and the final award will be made a few days later. The winner will receive a prize of $3,000, and the other four of the five final selections will receive respectively $2,000, $1,500, $1,000 and $500. It is planned to give soon after the final award an exhibition of all the 62 designs submitted in the preliminary contest.

THE MIXTURE OF MINERAL OIL AND CONCRETE is declared to be a possibility and the resultant a desirable compound by Mr. Albert Moyer, Assoc. Am. Soc. C. E., as a conclusion from a series of experiments he has recently conducted. This is along the same line as the tests now being carried on in the U. S. office of Public Roads, as reported by Mr. L. W. Page (Eng. News, Dec. 23, 1909, p. 710). The tests made by Mr. Moyer were as follows:

Neat cement briquettes were made with the water slightly in excess of that usually required, after which 10% of oil petrole was added. ("Oil petrole" is a white non-volatile petroleum product of about the consistency of melted vaseline). Pats were made of 1 part cement, 3 parts sand mixed with water, after which 10% of the same oil was added. These pats were about 2½ ins. in diameter and ¼-in. thick.

As soon as made they were left in dry air, the initial and final set was found to be normal. They were never immersed in water but remained in dry air for several weeks. No cracks occurred and they became so hard and strong that these pats, a ¼-in. thick, were very difficult to break by the use of the fingers and thumbs. After remaining in dry air for three weeks, they were put out in freezing temperature for three days, and again placed in dry air over the radiator. No cracks or checks occurred.

After remaining in dry air for a month, a test for absorption was made. A broken pat was weighed dry and found to weigh 94/64 oz. It was then immersed in water for several hours. Upon removal from the water, the surface water was quickly removed with blotting paper, the pat immediately weighed and found to weigh 99/64 oz. Only 5/64 oz. of water was absorbed.

In general the effect of the oil inclusion was to delay the initial and final sets, but up to a 15% oil content not enough to prohibit its use, and to reduce the tensile strength, brittleness, and tendency to craze or hair crack from surface contraction or expansion. This, then, gives a material admirably adopted for a number of purposes where it is essential that a tendency to crack and craze be absent and where the strength of the material need not be great.

As yet the long time effect of the oil on the wearing qualities of the concrete has not been discovered, but tests are now in progress to determine this. Mr. Mayer states that an included oil forms an emulsion in the concrete so that the proved decomposing action of an oil applied to hardened concrete does not indicate a similar action of an included oil. It is perhaps only fair to say that this is only an expression of opinion.

To mix oil in the concrete Mr. Moyer gives the following directions:

Turn the mixer a sufficient number of times to mix thoroughly the cement, sand, stone and water. Then add 10 to 15% of non-volatile mineral oil. Turn again the same number of times as it requires to mix the concrete. The oil will quickly emulsify and become thoroughly incorporated in the concrete.

It should be remarked that the above tests and deductions apply entirely to a mineral oil and must not be applied to vegetable or animal oils.

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The usual winter scourge of typhoid has filled the hospitals of Montreal to overflowing and has again raised serious questions regarding the water-supply of the city. How long will it be before the Montreal authorities wake up to a sense of their responsibility and provide a water-supply beyond question? It has been alleged, we believe, that the water-supply is not responsible for these winter outbreaks, but that seems to be against the evidence. It is high time the facts were determined and the remedy applied. The city bids fair to be regarded as a danger spot to be shunned by travelers, as was Pittsburg for so many years. Residents of a typhoid-infected city, that is such of them as survive, tend to become immune to typhoid, but temporary sojourners who come from cities relatively free from the disease, are in great danger of infection.

An interesting innovation in city administration has been introduced in New York City by Comptroller Prendergast, the new head of the Finance Department. This is nothing more nor less than the use of time cards for the various subordinates in his department. It is said that the use of the cards will be extended to other departments of the city. According to reports, each city employee hereafter will have to enter on the card the time of his arrival and departure from the office. We understand that in addition some notation will be made regarding the kind of work which is occupying the time of each employee during the day.

This time card system will serve several important purposes. The most obvious of these is a check on the length of time actually worked each day. A secondary but more important object of the time card will be to ensure that the appropriations are really expended for the purpose for which they were made. This is a serious matter in a large city, which is supposed to be working on a very rigid system of confining expenditures within appropriations for specific objects. In the past it is reported that great liberties have been taken in putting men onto one class of work and drawing their pay from the appropriation made for an entirely different purpose.

Still another valuable purpose which may be served by municipal time cards—and this will appeal strongly to engineers—is their use for making up cost figures. It is well known that where some attempt at determining unit costs is made, salary charges are often not distributed at all, or are distributed in a very arbitrary manner. With the daily card system as outlined, it will be a simple matter to distribute the time of each man to the particular class of work on which he was engaged throughout a given period.

We shall expect to see the time card system tried in other cities. Its workings in New York and elsewhere will be watched with great interest.

Some of the reasons for appointing an engineer rather than a physician as head of a municipal city health department have been mentioned from time to time in these columns. Such appointments have been made in a number of small cities during the past five or ten years, particularly in New Jersey, where engineers have served as executive health officers. In Plainfield, Elizabeth, Montclair and Orange, these appointments, however, have all been to administrative positions under the direction of boards of health. Sanitary engineers have been appointed to subordinate positions in some of the larger cities, but, so far as we know, it has been reserved for Philadelphia to install an engineer as the independent executive officer of a single-headed bureau of health. As we noted in our personal column last week, Mr. John A. Vogelson, M. Am. Soc. C. E., has received such an appointment in that city. The appointment came from Director Neff, Head of the Department of Health and Charities. Mr. Neff is reported as having based his appointment of an engineer rather than a doctor upon the fact that very considerable and important part of the work of the Philadelphia Bureau of Health in the near future would be along the lines of sanitary engineering. Mr. Vogelson had occupied an important position in the Philadelphia Bureau of Filtration, and so was in close touch with some of the health departments of the city.

Another important instance of going outside the medical profession for the head of the health department of a great municipality has just been afforded by Mayor Gaynor, of New York City, who has appointed Mr. E. J. Lederle as Commissioner of Health. Dr. Lederle, as he is called through virtue of his Ph. D., has been reinstated by Mayor Gaynor to a position which he filled with great credit to himself, and value to the city a few years ago. His first appointment was really in the line of well-deserved promotion, as Dr. Lederle had been connected with the Health Department for many years as chemist. Since his former period of service, Dr. Lederle has gained a great variety of experience which will be of value to the city by his work in connection with sanitary laboratories which he established for private practice, and also as a member of the New York State Water Supply Commission and as Consulting Sanitary Engineer for the New York Board of Water Supply. Altogether his appointment is a fitting addition to the many excellent selections which Mayor Gaynor has made for heads of departments, as we noted editorially a week ago.

We expect to see in the future an increasing number of selections from the ranks of engineers, chemists and bacteriologists to head municipal health departments. The facts are, that modern health-protective work has developed far beyond the capacity of the practicing physician. Doctors there are, of course, who by their breadth of interest and devotion to the public good since leaving their medical colleges, have done much to qualify themselves for the position of health officer, but as a rule, physicians are appointed to this position from a mistaken notion on the part of the public that their training and experience has eminently fitted them for it, and that there is no other professional class to draw from. Instead, as we have often pointed out, very few of our medical schools are able to do more than fittingly prepare their students for the practice of private medicine and scarcely any of them give adequate instruction in public health-protective work. The schools of sanitary engineering, chemistry and bacteriology do much better in this respect, although no one would maintain that they afford the ideal training for the prospective health officer.

It should always be remembered, when considering the desirability of selecting technically-trained men for administrative positions, that while technical training along the lines of the administrative work to be taken in hand is of great importance and value, yet the first requisite is administrative ability. This is particularly true in the case of a great city where the work of administration must of necessity be divided between many administrative subheads, each of whom may be chosen on account of his training and experience in the particular line of work which he is to administer. But where technical training is taken into account as a prerequisite of an appointee, then it is, of course, desirable that the person who makes the appointment should have clearly in mind the class of work which is to be administered, and the kind of training which would best fit the man to carry on that work. Previous experience in the same line of administration should, of course, always count, as it unquestionably did with Mayor Gaynor in the reappointment of Dr. Lederle.

Municipal ownership of street railways has made but little headway in the United States or Canada, although widely practiced in Great Britain. A few minor examples might be cited in both American countries, but as a rule even they have been the result of peculiar circumstances. Moreover, the chief examples—the New York and Boston subways—are not under municipal operation.

In view of the foregoing facts, the popular vote of San Francisco, on Dec. 30, 1909, in favor of a municipal street railway line is of much interest. The vote is all the more significant because it is the fourth one in a half dozen years on the same general proposition and was almost 3 to 1 this time. The three earlier votes, while giving a clear majority, were below the necessary two-thirds. Moreover, they were light votes, since the majority at each election was about 15,000. At the last election, 31,185 votes were cast in favor of the proposition and 11,694 against.

It is interesting to see what is proposed. As we understand it, a 25-year franchise for what is known as the Geary St. line, expired in 1903. This line comprised nearly four miles of double-track cable railway, beginning at Geary and Market Sts., near the Market St. Ferry. When the franchise expired, the city authorities, in accordance with the spirit of the then new city charter, called an election on a municipal ownership proposition. The estimated cost and corresponding bond issue is reported as having been increased from $750,000 at the first election, 1903, to $1,950,000 at the third election, in June, 1909. We presume the increase was due to changes in the plans for the road. At the election which has just proved successful, the proposed $1,950,000 for the Geary St. line was increased by $120,000 for a connecting link from Geary and Market Sts. to the Market St. Ferry, leading to Oakland, Berkeley and other points on the mainland.

The campaign for and against the bonds was hard fought and bitter. It was alleged, on the one hand, that the bonds must be carried in order to help break a monopoly that was defying the city and mistreating both its patrons and employees. On the other hand, it was alleged that the proposed municipal railway line would be a waste of public money, both because of extravagance in construction and operation and because as an isolated line out of harmony with the company system good service could not be expected. The Market St. expenditure, it appears, was proposed at the last election to meet possible difficulties incident to getting passengers between Geary St. and the ferry.

Notwithstanding all the arguments advanced by the Merchants' Association, the Chamber of Commerce and various civic organizations, and the alleged threats of the street railway interests, the bonds were carried by the heavy majority already stated. Of course that does not ensure construction, but the proposition was endorsed by the outgoing administration, headed by Mayor Taylor, and has the approval of the union-labor administration which has just come into office, headed by Mayor McCarthy.

Whatever may be the merits or demerits of the case, the people of San Francisco appear to have had a growing conviction of the desirability of trying municipal ownership of street railways. They evidently feel that they have been mistreated and that municipal service would prove more satisfactory. Perhaps a trial of the plan would teach both the people of San Francisco and the owners and managers of the existing street railway system some highly useful lessons.

The system of hauling trains or cars up steep grades by cable hoisting plants dates back to the early days of railways, and in those days it was applied to many comparatively easy grades, due partly to the insufficient power of the small locomotives and partly to the lack of understanding of the powers and possibilities of the locomotive engine. Two interesting cases where the cable system is still employed in this country as being more efficient and economical than locomotive traction on a longer route of easier grade were described in our issue of March 25, 1909. On another page of this week's issue we publish a paper comparing the locomotive and cable haulage systems for operating the trains of ore cars at open-mine excavations. The advantages appear to be largely on the side of the cable system, in regard to both the economic and operating conditions, and it would be a matter of special interest if some mining company having workings of this kind should break away from the conventional practice of locomotive traction and adopt the old system of cable haulage as modified for modern conditions. It will occur to many readers in this connection that contractors on earth excavation work often employ hoisting engines and cables to lift loaded cars out of a pit.

Mr. Gifford Pinchot, Chief of the U. S. Forestry Bureau, was dismissed from office by President Taft on Jan. 7. The reason for the action, to quote from President Taft's letter of dismissal, was "an improper appeal to Congress and the public to excuse in advance the guilt of your subordinates before the President could act and against the President's decision in the Glavis case before the whole evidence on which that decision was based should be considered."

It is exceedingly unfortunate that an official of such great usefulness as Mr. Pinchot should be lost to the public service. The country owes to him an immeasurable debt. On the other hand, it must be freely admitted by Mr. Pinchot's best friends, that his latest appeal to the public, through Congress, as set forth by the President in his letter of dismissal, was a breach of discipline which left the President no alternative but to issue the order for his removal.

We say this wholly without reference to the merits or demerits of the charges brought against Secretary Ballinger by Mr. Glavis. These charges are to be sifted by a Congressional Investigating Committee. It is now definitely established by Mr. Pinchot's own frank statement that two of the subordinate officials in his department aided Mr. Glavis in the formulation of the charges. That their action was a breach of good discipline in the public service is admitted by Mr. Pinchot himself. That Mr. Glavis and the Forestry officers believed they were rendering a public service in calling attention to a threatened alienation of public lands of great value we see no reason to doubt. Whether their fears of danger to the public welfare were justified or whether they were raising a needless cry of "wolf, wolf!" it is for the Congressional Investigating Committee to determine.

Secretary Ballinger and the United States Reclamation Service.

As those of our readers who follow the newspapers are aware, the administration of the Interior Department by Secretary Ballinger is to be investigated by a joint committee of Congress. One of the bureaus which will doubtless receive the careful attention of the investigating committee is the Reclamation Service, Secretary Ballinger's policy toward it, and his intention, publicly expressed, of making radical changes in it.

This matter is one of so much importance and interest to the engineering profession that it has seemed our duty at this time to publish some of the events which have caused grave dissatisfaction in the Reclamation Service during the past few months.

It is hardly necessary to say here that the engineering profession, as a whole, throughout the United States, takes a peculiar pride in the Reclamation Service. The profession is proud of the Reclamation Service because it is an object-lesson, proving that it is possible to build up in the Government service an efficient organization of civilian engineers. And this means more than the average reader may realize.

It has been for many years a stock argument against the creation of an organization of civilian engineers to carry on our Federal public works that no such organization could be made efficient. Politics would creep in first, and graft would creep in after it and the organization would soon become honeycombed with corruption. The same stock argument is brought forward whenever it is proposed to entrust any piece of work to the Government rather than to private enterprise.

The success of the Reclamation Service has been an effective reply to such arguments. It has been demonstrated that the engineering profession *is* capable of organization for the efficient conduct of great enterprises; that it is not without executive ability to couple with its technical knowledge.

And it is only giving credit where credit is due to say that the engineering profession and the country is most largely indebted for the success of the Reclamation Service to Mr. Frederick H. Newell, its Director, and Mr. Arthur P. Davis, its Chief Engineer.

Just how great has been their load of responsibility, and just how difficult are the problems with which these two engineers have been dealing, few outside the Reclamation Service are aware. It may be thought, possibly, that the Reclamation Service has had merely the simple task of building a few dams for water storage, and that any engineer experienced in dam construction could have carried on the work with equally satisfactory results.

The fact is—and we wish to emphasize it—that the leaders of the Reclamation Service are entitled to quite as much credit for their success in *administration* as for their success in solving purely engineering problems. They have not merely had to build dams and canals and tunnels and power plants on irrigation projects scattered all the way from Arizona to Montana. They have had to deal with complicated questions concerning land and water rights, concerning the distribution and utilization of water, concerning power generation and distribution, concerning the settlement and improvement of the reclaimed lands. Last, but not least, the Reclamation Service has had to comply with the complicated statutes and usages concerning work carried on by the Federal Government, and at the same time has had to steer clear of all the pitfalls due to political intrigue and the machinations of those who seek to use the service improperly to further private business interests.

This is a snap-shot picture of some of the difficulties which have had to be met in the administration of the Reclamation Service. These difficulties, we believe (and we have unusual opportunities for observation), have been handled with unusual ability.

How great a success has been attained can be realized best, perhaps, by a brief retrospect.

Ten years ago irrigation work in the West was at a standstill. The water in the streams available without storage had been mostly appropriated. So many private irrigation companies had come to grief that irrigation securities were unmarketable. While there were many successful irrigation works in operation, the failures of irrigation companies had been so numerous and so conspicuous as to wholly discredit irrigation with the investing public.

It was this situation which led the West to unite in a demand that the Government undertake the work of reclaiming the arid lands, and the result of this demand was the establishment of the Reclamation Service.

To-day, as a result of the successful work of the Reclamation Service, irrigation enterprise has been restored to public confidence. Private concerns are to-day embarking in irrigation work and are able to market their securities because of the able engineering and administrative work of the Reclamation Service. But out of this situation has arisen a certain antagonism toward the Reclamation Service. The promoters who are in favor of the conservation of natural resources, provided they can conserve the resources to their own profit, are jealous of the Government's irrigation work. And this jealousy is perfectly natural.

Inevitably, the private irrigation systems and the Government projects are in competition. The settlers under a Government project have no interest or profits to pay on the money invested to reclaim their lands. They have not the uncertainties regarding changes in financial control and their results that the settlers have on a tract reclaimed by a private corporation.

The promoters of private irrigation schemes are restive under this Government competition. They voice a demand that the Government should leave the best and most favorable locations for irrigation work to them, and confine the Reclamation Service to projects which do not promise any profits.

But if the Service accepts this and undertakes a project where the cost of reclamation is greater than the value of the land after water is applied, settlers will not come in, and the whole project will be a failure. Naturally, therefore, those who have the welfare of the Reclamation work at heart wish to secure as favorable sites as possible for carrying out the work, and there is a conflict of ideas between those who wish to have the Reclamation Service so administered that it shall not interfere with their schemes for making money and those who urge that the settlers under Government projects ought to have the benefit of the most favorable opportunities for irrigation that Nature affords.

There has been, too, without doubt, more or less jealousy of the Service in political circles, because there has been no chance to make political capital through appointments and influence in connection with the large expenditures for irrigation. The Reclamation Service was placed under Civil Service rules by President Roosevelt, and while this has to a certain extent hampered the work of administration, it has kept its personnel free from entangling alliances. The engineer who gained his appointment and promotion on his merits and who was assured of the confidence and support of his superiors has been able to treat all contractors fairly and has felt safe in turning down requests for improper favors, even though they were backed by the local political leader.

We have thus reviewed the situation in the Reclamation Service, to make clear to the reader how important it was to the Reclamation Work what sort of man was selected to administer the Interior Department, and how important it was to certain private interests to have the Reclamation Service so conducted that the promoters and speculators in land and water rights would get the fat, while the Government would take the lean.

We have not the slightest doubt that President Taft, in selecting Mr. Ballinger for the Secre-

taryship of the Interior, believed, and still believes, that the Secretary is devoted to the public interest, and that he possesses the executive ability necessary to efficiently discharge the duties of his office. There is no question, either, that Secretary Ballinger's public utterances on conservation and the text of his annual report give no cause for criticism on the part of those who stand for the conservation of natural resources in the interest of the people as a whole.

The question that is to be settled is in the first place whether Secretary Ballinger is actually and heartily on the side of the people and whether he possesses the executive ability and capacity necessary for the efficient discharge of his high office. The one of these is important as the other. A blunderer, or a man of small caliber at the head of the Interior Department, might have the public welfare sincerely at heart and yet do irreparable damage. This question must be settled, of course, by the Secretary's official acts; and it has seemed to be our duty to call attention to a few of them and their results.

We have explained above that the appointments and promotions in the Reclamation Service are made under Civil Service Rules. It need hardly be said that the chief object of the Civil Service system is to take the Government business out of politics. We are well aware of the valid grounds which engineers often find for criticising the civil service system; but all candid critics admit the great value of the system in eliminating political influence as a factor in appointment to office.

Not long after Mr. Ballinger became Secretary, we are credibly informed, he ordered that an engineer who had been Chairman of a State Campaign Committee, be appointed to a $3,000 position in the Reclamation Service regardless of Civil Service regulations. The appointment was not made, fortunately, as the engineer in question secured a better paying position through some of the private interests to which his political services had been useful. The news of the attempted appointment, however, became known among the engineers of the Service; and what may have seemed to Mr. Ballinger a trivial matter was to them very serious. Under the previous administration they had come to consider that security in their positions and promotion depended on their own ability and achievement. But if the new Secretary of the Interior was to appoint men for political services, what man of them was safe? Very likely Secretary Ballinger himself never realized the widespread harm to the Service resulting from his proposal to appoint a political engineer to a $3,000 position; but if he did not realize it, that very fact seems to us to evidence lack of executive ability.

It was not long, however, before the engineers of the Service received a jolt that made them far more anxious. In various newspapers in the West and elsewhere there was published early in June a statement regarding the intentions of Secretary Ballinger respecting the Reclamation Service, which was apparently made public with his approval, and was at least not contradicted by him. The important part of that statement is contained in the following quotation from the Portland "Oregonian" of June 8:

> Unless there is a change of program, F. H. Newell will retire from the office of Director of the United States Reclamation Service some time during the coming fall.
>
> The situation, in brief, is this: While Mr. Ballinger finds no fault with the engineering work of Mr. Newell, but has respect for his ability as an engineer, he inclines strongly to the opinion that he can find a more satisfactory administrative officer to take general supervision of the Reclamation Service—to become its business manager, so to speak. He recognizes that Mr. Newell's abilities lie along the lines of engineering rather than administration, and that fact inclines him to the opinion that a change should be made. By retaining Mr. Newell as an engineer officer, he can save to the Reclamation Service the valuable advice of the present director on all problems affecting construction, while by appointing a new director he believes he can overcome those unsatisfactory features of administration which he disapproves.
>
> The inclination of Mr. Ballinger to urge a change is not due to the activity of politicians who have quarreled with Mr. Newell; it is not due to the recommendation of any one. It is based on his own experiences with the Reclamation Service since he became Secretary of the Interior. As a Government machine, that Service is not running as smoothly as might be desired; there is not entire harmony between the Service and the present head of the Interior Department. On many details of administration the Reclamation Service, under previous secretaries, has followed a policy that does not meet with the approval of Mr. Ballinger. He insists that his ideas be followed by all branches of his department.
>
> Mr. Newell has not worked at cross-purposes with Mr. Ballinger, but rather at his direction. Nevertheless, there is not that same bond between the Service and the present Secretary that was noted between the Service and Secretary Garfield. Under Mr. Garfield the Reclamation Service took the lead, and the Secretary approved; now the Secretary takes the lead, and the Service must follow his directions.

Every Reclamation Service engineer who read that interview, we have little doubt, said to himself, "If Newell can't hold his place under this administration—Newell who has built it up and made a success and done vastly more valuable service for it than ever I can hope to do—what chance have I of holding my job?"

If Secretary Ballinger inspired that interview, or if he permitted his intentions regarding Mr. Newell to become public at that time, he thereby, it seems to us, showed lack of discretion and executive ability. The competent manager of an enterprise who finds it advisable to make a change in the heads of departments under him, is careful to do it in such a manner as to least affect the morale of subordinate employees.

This, however, is a trivial matter, compared with the main issue—the removal of Mr. Newell from his position as Chief of the Reclamation Service. Why should the Secretary desire to depose Mr. Newell? Analyzing the statement in the interview above quoted it appears to simmer down to the assertion that "there is not entire harmony between the present head of the Interior Department and the Service." "Secretary Ballinger takes the lead and the Service must follow his direction." There are insinuations, in the above interview, that Mr. Newell has not been successful as an administrator; but there is not, nor has there been to this day, any specific evidence to contradict the opinion generally held, we believe, throughout the engineering profession that Mr. Newell has not only been a successful executive officer but that he has shown most conspicuous ability as an administrator.

Actually Mr. Newell has had far more experience in executive work than Mr. Ballinger himself.

The engineering profession takes great pride in seeing its members advanced to positions involving executive responsibility as well as technical ability. It will not stand for a man who is incompetent; but there will be most emphatic protest, we are sure, against Mr. Ballinger's proposal to take away executive responsibility from Mr. Newell unless and until it can be shown that the motive for such a change is the public welfare, and not merely Mr. Ballinger's personal prejudices or purposes.

Suppose, for the sake of argument, that Mr Ballinger is really a man of sound judgment, animated solely by zeal for the public welfare. In that case it seems to us he would have first of all ascertained conditions in the Reclamation Service, taking care not to interfere with the work already in progress and the efficiency already obtained before undertaking any radical "reform" measures. But Mr. Ballinger actually started in at the very outset of his administration to make Mr. Newell's position as disagreeable and uncomfortable as possible *before he could have had any knowledge whether Mr. Newell was efficient or not.* Some will interpret this as circumstantial evidence that Mr. Ballinger desired to get rid of Mr. Newell and tried to bring about his resignation in this way rather than incur the unpopularity that would result from removing him from office.

There is, however, another explanation. Mr. Ballinger appears to have taken office with the notion that the previous Administration had been all wrong; that its conspicuous successes in the Reclamation Service had been made in disregard of the statutes and that it was his foremost duty to ferret out past blunders and make such removals and changes in the personnel as would ensure better results in the future. This seems to be the most charitable explanation of his suspicious and hostile attitude from the outset toward those officers of the Service who had been considered most able and efficient under President Roosevelt and Mr. Garfield.

Perhaps Mr. Ballinger assumed this attitude in good faith; but even so, its results were most disastrous to the Service. The effect of such widely published interviews as that quoted above was to cause some of the most competent engineers in the Service to resign their places and accept positions at better salaries with private companies. Can Mr. Ballinger blame them for so doing? Where and in what way have they had any assurance from him that merit should continue to be the sole requirement for an engineer to be secure of his position and of promotion?

The engineers in the Service have been content to work for smaller salaries than those received by engineers employed by private companies, because of the organization and the standards that have hitherto prevailed, the permanence of the work and the confidence of support by their superiors. Does Mr. Ballinger appreciate the value of such an *esprit de corps* to the Service? Has he done anything to perpetuate it?

But the harm to the Service has not stopped here. Secretary Ballinger's attitude of suspicion has caused months of delay in the letting of contracts and the prosecution of work on some of the projects. He has refused to approve of work until he could satisfy himself that everything was all right. Whatever excuses there may be for such vigilance in a public official where vigilance is needed and justified, it has been most unfortunate in its effect on the work. The delays which he has caused have increased the cost of works by hundreds of thousands of dollars. This added cost falls not upon the United States Treasury, but upon the poor settlers on these irrigated lands in the shape of an added price for their farms and water rights.

And now what has Secretary Ballinger unearthed in the Reclamation Service to justify his attitude of suspicion? We are reminded of the mountain which labored and brought forth a mouse, for up to date the only important public act of Secretary Ballinger in reversal of the policies of the previous Administration has been to stop the cooperative work of the settlers under the Reclamation projects in the construction of irrigation ditches. We fully explained the absurdity of this ruling of Secretary Ballinger in our issue of Sept. 30, 1909. It need only be recalled here that the sole ground on which this beneficent cooperative work was stopped was the legal technicality that there was no specific authority in the statutes for carrying out work in that particular way. The cooperation was for the manifest benefit of the settlers, and caused no loss or risk of any sort to the Government. The stoppage of this work on a technical point of law is all that Secretary Ballinger has to show in justification of his attitude of hostile suspicion and the positive injuries to the Service which we have above set forth.

And now why did Secretary Ballinger upset this cooperative work?

Has his experience as a lawyer made him a stickler for the letter of a statute, regardless of its spirit? Does he really believe in enforcing such narrow interpretations of the law as that which stopped the settlers from building irrigation ditches on Government projects? It is well known that there are men in the legal profession, who are excellent lawyers; but whose close adherence to precedent and detail, unfits them to fill positions demanding large executive ability, where prompt decision and wise generalship are essential.

But were there other possible reasons for this marvellous stickling over the letter of the law? It is a fact that the immediate effect of this change in policy by Secretary Ballinger was to stop the work on a Reclamation project at Grand Junction, Colo. The lands to be reclaimed at that place are enormously valuable when water is ap-

plied, some of the tracts selling for two or three thousand dollars an acre. We are credibly informed that after it was decided to carry out Reclamation work at this point on the cooperative plan, a private irrigation company concluded that it could make good profits if the Government would withdraw in its favor. The result of Secretary Ballinger's ruling forbidding further cooperative work was that the Government did withdraw, to the great disappointment of the settlers, who had sought to have the Government carry out the project.

To counteract the setback to the Reclamation Work given by the stoppage of co-operation with the settlers, Secretary Ballinger is now recommending a bond issue of $30,000,000 to supplement the Reclamation Fund. Before any such bond issue is made, however, it ought to be definitely established what the policy of the Reclamation Service under Secretary Ballinger is to be. If it is his policy to leave the profitable locations to private companies and devote the Government funds to the places that private capital will not touch, then there is grave doubt whether such bonds, if secured only on the revenues from the completed works, would attract investors. And that doubt becomes still more serious in the absence of assurance that the high quality of engineering work that has prevailed in the past is to obtain in the future.

Engineering News has no politics. We have undertaken to speak in this case because: it seemed to us our duty to defend the reputation of a distinguished engineer, to defend the reputation of the profession as successful administrators, to defend the Reclamation Service against any attempt to make it a political organization. More than all else, to defend the public interest.

We shall wait, as the public will wait, to see what justification Secretary Ballinger can make of his official acts and what methods he can adopt to make the public as trustful of his intentions and his ability, as it is now, we believe, distrustful.

There is little doubt that the Secretary will attempt to justify his attitude toward the Service by allegations that mistakes have been made in the Reclamation Service work. But such charges ought to be carefully sifted before acceptance. The question is not whether errors have been made. No enterprise involving the expenditure of ten thousand or a million dollars—much less fifty million dollars—was ever carried out without more or less errors. The question is whether any other administrator in Mr. Newell's place would not have made greater errors. The question of even greater practical importance is where an engineer could now be found of such executive ability, such familiarity with irrigation work and such knowledge of the complicated problems of the Reclamation Service, that he could take Mr. Newell's place and do anything like as well as Mr. Newell himself is doing.

That question, we submit, is of far greater moment to the country than the securing of a head for the Service who will "carry out Mr. Ballinger's ideas." Without doubt Mr. Ballinger has the legal authority to impose upon the Service any ideas he may have and to change the personnel to conform to those ideas; but equally without doubt, Mr. Ballinger holds his position as a trustee, sworn to protect the public interest. He has no right to govern the Reclamation Service by whim, by personal prejudice, or by any other consideration or motive whatsoever than the public interest. Can he convince the public, can he convince the engineering profession, of his good faith on the one hand and of his competency on the other?

LETTERS TO THE EDITOR.

A New Use for a Cold Snap.

Sir: Anent the opening of the Manhattan bridge, an Ohio newspaper, on Dec. 30, printed the following:

> The roadways and promenades are all ready, and a decided drop in the temperature to make the structure more elastic, is what has been awaited as a signal for the formal ceremonies incident to its opening.

New York, Jan. 6, 1910. "Reader."

Continuity in Reinforced-Concrete Beams.

Sir: In your issue of Oct. 28, 1909, p. 470, there appeared some criticisms upon the writers article published in your issue of Sept. 30, 1909, p. 344. Since the contributors of these seem to have overlooked some important points in the article of Sept. 30, a little explanation would appear necessary.

Mr. Goodrich, in his letter (Eng. News, Oct. 28, 1909, p. 470), denounces as "entirely illogical and erroneous" the method of not considering the concrete in tension as adding to the stiffness of the beam. To be sure, that portion of concrete near the neutral axis of a section subjected to bending does not crack and hence adds very slightly to the stiffness of the beam. Moreover the concrete on both sides of the neutral axis at points of contraflexure in a continuous beam does not crack at all and hence should be considered as adding to the stiffness at that point, but, since by the Theory of Least Work the work done by bending near and at the points of inflection is very small, this would not appreciably affect the results given. The writer believes the method of determining the stiffness ratio as outlined by Mr. Goodrich to be erroneous for the following reasons:

(1) Tests of actual beams under known loading show that, for working stresses in the steel, the concrete in the tensile side of the beam does not add appreciably to the stiffness of the section subjected to bending, but that both the position of the neutral axis and the stresses in concrete and steel agree very closely with the theoretical results obtained by considering all tension in the concrete as ineffective. As perhaps the latest proof of this the writer would refer the reader to a series of tests made by Mr. R. P. Davis as reported in "The Engineering Record," Nov. 6, 1909, p. 517.

(2) Even on the assumption that all the concrete in tension adds to the stiffness of the beam, assumed as an example in the writer's article of Sept. 30, if 80 ins. be the effective flange width between supports, the effective flange width over the supports would be only about one-half of 80 or about 40 ins. since the effective flange width decreases directly with the decrease in distance between points of inflection, and the stiffness ratio would be far below the value (0.78) given by Mr. Goodrich.

Further, if the stiffness ratio of 0.78 as given by Mr. Goodrich be assumed as correct, the moment coefficients as given (1/22 and 1/9, respectively) could not possibly be correct, for since Mr. Goodrich refers to a beam with both ends fixed, it is evident that for stiffness ratios of less than 1.0 the negative moment coefficient could not be greater than 1/12, as against the 1/9 given by him.

Mr. H. W. Schlinz in his letter (Eng. News, Oct. 28, 1909, p. 470) seems to have overlooked the fact that the reason the writer recommended more steel between supports than was necessary, was in order to reduce the deflection and relieve the high stresses at the support. As to the matter of the wide flanges assumed by the writer, a little investigation will show that if the flange width be decreased by one-half of that given, the lever arm of the moment couple (jd) will be changed only a very slight amount and the practical results obtained still very close to those printed.

The writer in his article wished to bring out the following points:

(1) That the fiber stresses at the supports of a continuous reinforced-concrete beam are, under ordinary conditions, very high and should receive more attention than is ordinarily given to them. This point needs no further treatment here.

(2) The use of more steel than is necessary between supports, thus making a very stiff section at that point, greatly relieves the stresses at the supports, makes the load-carrying capacity of the beam larger, and proves to be economical. To illustrate this point the writer submits herewith Fig. 1, which illustrates the decrease in negative bending moment effected by an increase in tensional steel between supports. The point to be borne in mind is that the negative bending moment at the support limits the load-carrying capacity of the beam. Hence anything we can do to reduce this negative moment increases the safe load of the beam and is economical up to certain limits. The writer believes this increased stiffness between supports can be best obtained in practical design by the use of a higher bending moment between supports than would be required by the actual stresses at that point.

(3) If the beam be figured for a bending moment of less than $1/8\ Wl$ between support, disregarding negative stresses, these negative stresses at the support often reach dangerously high figures. To show this better the writer submits Fig. 2. All curves of this figure are computed from the section used by the writer in his article of Sept. 30 and reproduced in Fig. 2 for convenience. If we design the beam for $M = 1/10\ Wl$ between supports regardless of stresses at supports we note from Fig. 2, curve A, that, whatever the amount of steel passed over the supports, the stresses there are dangerously high and far outside of reasonable design. If, however, we design the beam for $M = 1/8\ Wl$ between supports (as we would do in case of a simple freely supported beam) we find (Fig. 2, curve B) that, if about 2/3 of the steel used between supports be passed over the supports at the top of the beam, the induced negative stresses are no longer in excess of the safe allowable stresses assumed, but that the beam is working under its maximum efficiency. This method of calculating the reinforcement steel is rapid, safe, and efficient.

In the second edition of "Concrete, Plain and Reinforced," by Taylor & Thompson, p. 439, we are recommended to reinforce continuous beams for $1/12\ Wl$ both at supports and between supports. This practice is there shown to be safe for all classes of fixed end beams, and the writer believes it to be the best possible method to follow when a haunch or bracket is used to strengthen the stem of the beam at the support. In the ordinary beam, however, the writer believes it is possible to secure a more desirable arrangement of steel by reinforcing for as high as $1/8\ Wl$ between supports and passing 2/3 as much steel over the support as was used in the center of the span. The explanation is that this stiff mid-section reduces the stresses at the support, assuming the same section and reinforcement there, about 15% below the stresses when the beam is designed in the middle for $1/12\ Wl$, so that a given load would produce lower stresses than would be experienced in the latter case. Structurally either method would be entirely satisfactory.

The advantages of using a very stiff section between supports, as is obtained by the use of $M = 1/8\ Wl$, are:

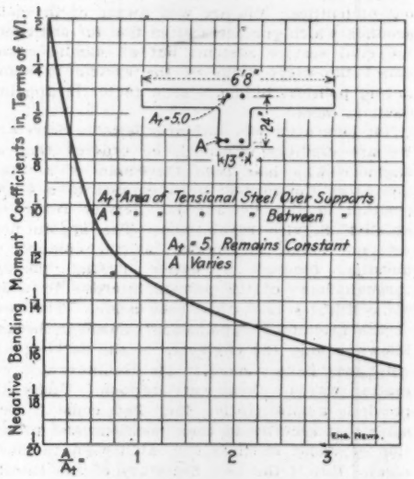

Fig. 1. Curves Showing Values of Negative Bending Moments for Different Ratios of Tensional Steel Between and Over Supports.

(Laid out by Mr. R. E. Spaulding.)

(1) Greater efficiency of the beam due to relief of the high stresses at the supports.

(2) A distribution of steel in the beam which is easily effected in practice (running 2/3 of the steel over the supports as is used between supports).

(3) A system which is much less affected by variations of loadings in different spans, non rigidity of supports, unequal settlements, etc., than is a beam of more nearly constant moment of inertia.

If a haunch or bracket be used to stiffen the stem of the beam at points of support, these advantages partly disappear and it proves more satisfactory to design the reinforcement for the beam on a basis of $M = 1/12\ Wl$ both at and between supports, as indicated above.

R. E. Spaulding.

Worcester Polytechnic Institute, Worcester, Mass., Nov. 30, 1909.

[The curve presented by Mr. Spaulding in Fig. 1 proves only that there is a certain decrease in negative bending moment at supports due to the original assumption of an excessive bending moment, with consequent excessive tensile reinforcement, at the middle of the beam. It does not show that it is more economical to make this assumption. The curves in Fig. 2 show that for assumed bending moments lower than $\frac{1}{8}\ W\ l$ at the middle of the beam, extra compression area, either in the form of excess reinforcement or a concrete haunch or bracket, must be provided to bring the negative moment of resistance up to the induced negative bending moment. It also shows that if the $\frac{1}{8}\ W\ l$ assumption is made, within a limited range ($p_t = 0.56$ to $0.80\ p$) the negative moment at the supports is sufficient.

In these curves the question of economy is neglected.

The chief criticism of the method proposed by Mr. Spaulding has been that the bending moment at the support is reduced so slightly by stiffening the beam at the middle that the reduction in steel at the support through this means will not balance the increase at the middle required for the stiffening. That is, though the method may be safe, it is not economical. This criticism was very effectively maintained for rectangular beams by Mr. P. E. Stevens in the Transactions of the American Society of Civil Engineers, Vol. 60, 1908, p. 496, but the subject of T-beams was not there discussed. As noted above, the economy of the method for T-beams is not proved by Mr. Spaulding's curves in this issue.

There follows a communication from Mr. Sanford E. Thompson upon this subject.—Ed.]

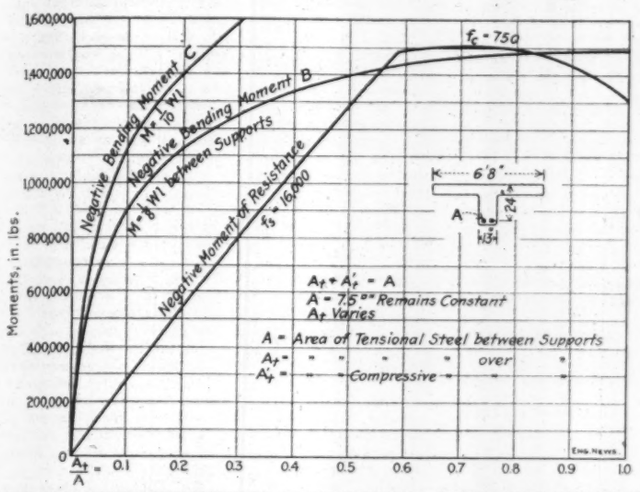

FIG. 2. CURVES OF MOMENTS OF RESISTANCE AT THE SUPPORTS AND OF INDUCED NEGATIVE BENDING MOMENTS FOR VARYING DISPOSITION OF STEEL BETWEEN UPPER AND LOWER SIDES AT SUPPORTS AND CONSTANT AREAS IN MIDDLE.

(Laid out by Mr. R. E. Spaulding.)

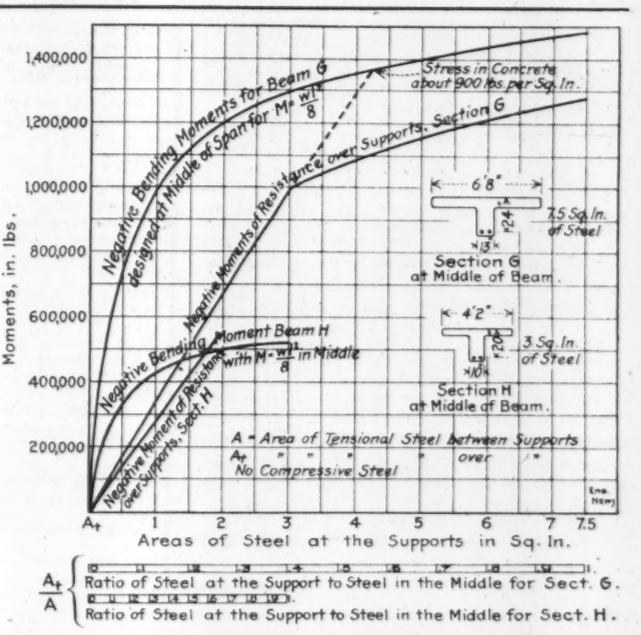

FIG. 3. CURVES OF MOMENTS OF RESISTANCE AT THE SUPPORTS AND OF THE INDUCED NEGATIVE BENDING MOMENTS FOR VARYING AREAS OF STEEL OVER THE SUPPORT AND CONSTANT AREAS IN MIDDLE. (NO COMPRESSIVE STEEL AT SUPPORT).

(Laid out by Mr. S. E. Thompson.)

Sir: Reinforced-concrete beams running over several supports are cast either as monolithic or built with joints in the concrete between two day's work but with the steel continuous so as to bind the parts together and make them act as a unit. Beams thus necessarily continuous or fixed are subjected to positive bending moment in the middle of the span and negative bending moment over the support.

Formerly, and sometimes even now, such beams have been designed in the middle as simply supported, thus giving an excess of steel in the center of the span and but little if any steel over the support. The tendency to design in this way is accounted for by the fact that engineers unacquainted with the peculiarities of the new material attempted to apply to reinforced concrete the same rules and methods which they had been accustomed to use in steel constructions.

In economical design of reinforced concrete, on the contrary, an engineer may use the continuity of a beam to advantage, but it is essential to provide for both the positive and the negative bending moment. Unless this negative moment is provided for, cracks, unsightly if not dangerous, are almost sure to occur.

The design of continuous beams is governed largely by the bending moments chosen. The moments in continuous beams are dependent upon many conditions, such as the number of spans, method of loading and shape of beam, and can be accurately determined for special cases by mathematical treatment or by the use of influence lines.

In general practice, however, such as building construction, the loading is usually indefinite, and it is simplest to consider the loads as uniformly distributed, and this plan is usually followed in design except where the exact conditions of panel loading or concentrated loads are known in advance, as in bridge design.

It is thus possible for ordinary conditions to give general recommendations which do away with complicated mathematical computations and at the same time are safe and conservative.

At the present time the conclusion has been reached by many of the foremost authorities that for a continuous beam of any number of spans with fixed ends, the maximum positive moment in the middle of the spans and the negative bending moment over the supports may be taken as numerically equal to each other and represented by the formula

$$M = \frac{wl^2}{12},$$

where, adopting the customary American units

M = bending moment, in in.-lbs.
w = load uniformly distributed in lbs. per in. of length.
l = length of beam, in ins.

If the end of the end span of a continuous beam is simply supported or is not rigidly fixed, the center of this end span and the adjoining support between this and the next span should be designed for a moment

$$M = \frac{wl^2}{10}$$

Continuous beams of two spans with the ends simply supported would be designed for a positive bending moment of $\frac{wl^2}{10}$ in the middle of the span and a negative moment of $\frac{wl^2}{8}$ over the center support.

It is frequently difficult to determine whether the end should be taken as rigidly fixed, but consideration of the relative mass of the support and the beam in question and also the method of connection will aid in determining this. In any case, however, if the end of the beam is built monolithic with other parts of the structure, it should be reinforced over the top of the support in order to prevent cracks at this point, even if this is not relied upon in the rest of the design.

The maximum positive bending moment suggested is larger than the theoretical moment in a truly continuous beam with uniformly distributed load, which is $\frac{wl^2}{24}$. This excess is advisable, however, to provide for imperfect continuity of the beam, for any increase in bending moment due to the varying of the moment of inertia and for the effect of unloaded panels. In nearly all cases also the moment of inertia of the beam over the support is smaller than that in the middle, so that the negative bending moment with uniform load will be smaller than $\frac{wl^2}{12}$.

It is easy to comply with the above requirements in a rectangular beam, because it means simply to place as much steel over the support as in the middle of the beam. In a T-beam, the case is more difficult. In the middle of a T-beam the flange furnishes a large compressive area, while over the support the concrete in the flange being in tension is ineffective. The compressive area is much smaller than in the middle of the span, and it is frequently necessary to make up for this by means of steel placed in the lower or compressive part of the beam or by the deepening of the beam by forming a shallow tapering haunch. The maximum allowable compressive fiber stress in the concrete at the support may be increased over the fiber stress used in the middle of the beam for reasons discussed later on.

A common argument against reinforcing a continuous reinforced-concrete beam over the support for the full negative bending moment derived for beams with constant moment of inertia is the fact that this moment can be reduced by making the beam stiffer in the middle. However, the reduction in bending moment at the support due to the excessive stiffness in the middle is too small, in the judgment of the writer, to permit an economical design.

To prove this the writer refers to the article published by Mr. Ralph E. Spaulding in the issue of Engineering News for Sept. 30, 1909, under the title: "Moments in Continuous Reinforced-Concrete Beams under Uniform Loading," which should be welcomed by engineers interested in reinforced construction as an important contribution in a field as yet scarcely entered upon in this country.

This article takes into account the variation in the moments of inertia in reinforced beams, and although based on somewhat different assumptions,* gives results which substantially agree with studies made in the office of the writer in connection with the preparation of the second edition of Taylor & Thompson's "Concrete, Plain and Reinforced," which has just been issued. In this analysis it was shown that the difference between the bending moments in reinforced-concrete beams designed according to the suggestion put forth in the preceding paragraphs is inappreciable and reaches in extreme cases only about 10%.

Mr. Spaulding in the conclusion to his analysis assumes a section of a T-beam and computes the moments at the center and at the support for different proportions of steel over the support and in the middle. His results are duplicated in the following table and for comparison the moments for continuous beams uniformly loaded are also given.

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*Mr. Spaulding in his study assumed that the moment of inertia of a reinforced-concrete continuous T-beam is constant between the points of inflection, and there changes abruptly to another value which is constant for the ends of the beam.

This assumption neglects in all cases the value of concrete below the neutral axis because it is in tension and is liable to cracking. This, however, is nearly true only for the sections subjected to the maximum bending moments, where the tensile stress exceeds the tensile strength of concrete. For other sections it is not exact. It is clear that with the diminishing of the bending moment, the tensile stress in the concrete diminishes, and thus decreases the depth of concrete subject to cracking until it becomes zero at a point where the concrete is capable of sustaining the developed tension without cracking. In this uncracked portion of the beam the total section should be considered in determining the moment of inertia.

Hence, the variation of the moment of inertia throughout the beam may be represented by a curve with maximums at the points of inflection and minimums at the middle of the beam and the supports. The acceptance of such variations in moments of inertia would add to the exactness of the analysis, although, as already stated, the results figured in the writer's office in this way are quite close to Mr. Spaulding's results. It should be noted, however, that the writer does not agree with the conclusions reached by Mr. Spaulding relating to the proper way of designing reinforced beams. The reasons for this disagreement are set forth in the present article.—S. E. T.

	Moment at center.	Negative moment at supports.
Continuous beam, theoretical value based on a constant I..	$+\frac{1}{24}wl^2$	$-\frac{1}{12}wl^2$
Assumed T-beam‡ with two-thirds as much steel over support as in center of beam....	$+\frac{1}{19}wl^2$	$-\frac{1}{14}wl^2$
Same T-beam with one-third as much steel over support as in center of beam.....	$+\frac{1}{16}wl^2$	$-\frac{1}{16}wl^2$
Same T-beam with one-sixth as much steel over support as in center of beam.....	$+\frac{1}{14}wl^2$	$-\frac{1}{19}wl^2$

‡The section of this T-beam is given as section G in Fig. 3.

It is evident from this table that the negative bending moment over the support decreases with the decrease of the amount of steel, but it is also evident that the decrease in amount of steel over the support and therefore the consequent decrease in the moment of resistance at that point is larger than the decrease in the induced moment.

Suppose a beam is designed for $M = \frac{wl^2}{8}$ in the middle of the span and one-third of the steel is placed over the support, then there would be only enough steel there for about one-third of $\frac{wl^2}{8}$ or for approximately $\frac{wl^2}{24}$, while the induced bending moment would call for $-\frac{wl^2}{16}$, as given in the table.*

It is possible to show that for a T-beam reinforced with more steel than 2% of the area of the stem (including the portion of the slab directly above it), it is impossible even by using in the middle of the span an amount of steel required by a moment $\frac{wl^2}{8}$ to avoid in conservative design the necessity of strengthening the compressive part of the beam at the end by a haunch or by compression steel.

A diagram (Fig. 3) has been prepared to determine for what ratio of steel over the support to that in the middle of a beam designed for a bending moment $\frac{wl^2}{8}$, the moment of resistance is equal to or greater than the induced bending moment. The beam has been considered in all cases as reinforced with tensile steel only to resist the positive and negative moments. For a beam reinforced with compressive steel or with a haunch at the support, the moments of resistance would be larger than shown at the diagram, so that the curve of bending moments may intersect the curve of moments of resistance [as on Fig 2—Ed.], and a larger percentage of tension steel can be used without overstressing the concrete in compression. This, however, will not appreciably affect the relative economy of the cases considered.

As a basis for the diagram, two T-beams with different percentages of steel have been selected, the upper curves being based on a T-beam of the same section used by Mr. Spaulding, while for the lower curves a T-beam of a small section with a smaller amount of steel is used for comparison. The sections are reproduced in the diagram. Mr. Spaulding's original diagrams were used, in both cases for determining the ratio of the moments of inertia and bending moments required in computing the curves.

In the upper curves, for section G, it is evident that, whatever the ratio of steel at the center of the beam to that over the support, for safe working stresses in steel and concrete the moment of resistance is always lower than the corresponding bending moment.

The sudden break in the curve representing the moment of resistance of the larger beam is caused by the compression in the concrete reaching the maximum working value which is taken at 750† lbs. per sq. in., and from that point on further increase in the amount of steel increases the moment of resistance but very little. The dotted continuation of the curve represents moments of resistance governed by the steel with a corresponding compressive stress in the extreme fiber of concrete larger than 750 lbs. This dotted curve intersects the curve of bending moments at a point at which the area of the steel at the support, $A_s = 4.2$ sq. ins. and the ratio of area of steel at the support to area of steel in the center, $\frac{A_s'}{A_s} = 0.56$. This signifies, for the section considered, that for 4.2 sq. ins. of steel over the support the bending moment there is equal to the moment of resistance governed by steel in the top of the beam, and that the extreme fiber stress in concrete at the bottom of the beam is about 900 lbs. per sq. in. It is further evident from the diagram that to keep the stress in the concrete at the support within the working limit of 750 lbs. per sq. in., it will be necessary with the given design of section G to introduce some steel in the bottom of the beam or else form a haunch to give extra compression area.

From the lower curves, drawn for Section H, which is a T-beam reinforced with 3 sq. ins. of steel that forms only 1.5% of the area of the stem, the curve of bending moments intersects the curve of moments of resistance at a point corresponding to 1.72 sq. ins. of steel at the support. This steel forms 0.86% of the area of the stem, or about 56/100 of the steel used at the middle of the span, the same ratio as in the other section. The fiber stress in the concrete corresponding to this amount of steel stressed to 16,000 lbs. per sq. in. is 700 lbs. per sq. in.

As a conclusion from the diagrams, it may be stated that if a beam is designed at the middle for a bending moment of $M = \frac{wl^2}{8}$, it requires over the support about 56/100 of the steel used in the middle of the beam, or only about 14% less area of steel at this point than would be called for using $\frac{wl^2}{12}$. For working compression of the concrete at the support of 750 lbs. per sq. in. and 16,000 lbs. in the steel, the required percentage of steel is about 1%. It is thus possible to get along without any additional strengthening of the compressive part of the beam at the support only when the beam is reinforced at the middle with less than $1\% \div 56/100 = 1.8\%$ of the area of the stem.

Now let us consider the economical part of the proposition. The beam shown in the lower part of the diagram, Section H, designed as a simply supported beam at the middle of the span, would require

3 sq. ins. of steel in the middle and
1.72 sq. ins. of steel at the support,

while designed according to the method now being quite generally adopted of using $\frac{wl^2}{12}$ both in the center and at the support, there would be used

2 sq. ins. of steel in the middle and
2 sq. ins. of steel at the support.

To compare the cost in the two cases the volume of the steel must be considered. In determining the volume of steel, assume that the beam designed for a moment $\frac{wl^2}{8}$ in the middle is reinforced by rods the area of which amounts to 3 sq. ins. Of this amount 1.72 sq. ins. of steel are bent up and carried over the top of the support, the rest of the bars being horizontal in the bottom of the beam. The approximate volume of this steel may be taken as the area of the steel in the middle times the span in inches or $3 \text{ sq. in.} \times l = 3\,l$ cu. in.

In the other beam assume that half of the 2 sq. ins. of steel on the bottom is bent up and carried at the top over the support and that the rest of the reinforcement needed for the negative bending moment for 1 sq. in. is brought over from the adjoining spans. Then the volume of steel may be taken as the volume of the long rods, the area of which amounts to 2 sq. ins., plus the volume of steel carried over from the adjoining spans and extending to about 1/5 of the span from each support, or approximately $(2 \text{ sq. in.} \times l) + (2 \times 1 \text{ sq. in.} \times 1/5\,l) = 2.4\,l$ cu. in.

Comparing the steel in the two beams of equal load carrying capacity, it is evident that the beam designed for a moment $\frac{wl^2}{8}$ in the middle of the span requires $3.0\,l - 2.4\,l = 0.6\,l$ cu. in., or 25% more steel than the beam designed with $\frac{wl^2}{12}$ for both positive and negative bending moments.

On the other hand, some cities require that the beams and girders be designed in the middle of the span for a bending moment $\frac{wl^2}{8}$. In such a case to obtain a safe structure and at the same time one that is as economical as possible under the conditions, the beam may be designed with steel in the center up to about 2% of the area of the stem, and with about 6/10 of the area of the steel at the top over the support. Some steel of course should be carried on the bottom all the way through the beam.

As already stated, it is permissible to increase the maximum working compressive fiber stress in concrete at the support over that used in the center of the span. Reasons for this increase are as follows:

(1) The Joint Committee on Concrete and Reinforced Concrete recommend that the span length for beams and slabs be taken as the distance from center to center of supports, but not to exceed the clear span plus the depth of beam or slab. When taking this span the maximum bending moment occurs in the theoretical point of support inside the column or girder. At the edge of the column or girder the moment will be smaller than the maximum, and the true section will be subjected to smaller bending moment than that obtained from computation. (2) From inspection of the moment curve for a continuous beam, it is evident that the negative bending moment decreases very rapidly from its maximum at the theoretical point of support to zero at about one-fifth of the span, hence the stresses in concrete and steel decrease very rapidly also so that only a very short length of the beam abutting on a larger surface is subjected to the maximum stress, and the conditions are comparable with the loading of a very short prism, which will sustain a much higher load than a cube or a long prism. (3) Furthermore, the concrete is surrounded by a mass of steel and stirrups which tends to increase the ultimate strength as does hooping in a column. In a continuous beam the bond and shear stresses must receive their due attention. The decrease in the negative bending moment, as already stated, is rapid, and for that reason the bond stresses, which are proportional to the decrease in moments, are apt to be high, especially in short heavily loaded beams. Consequently, bars of diameter which could develop sufficient bond strength in the middle of the beam may not have sufficient surface area at the end of the beam, in which case a larger number of smaller bars or deformed bars may be used.

Diagonal tension, which is the resultant of shear and tension, reaches its maximum intensity in the tensile part of the beam, so that, to prevent pulling out, stirrups must be securely anchored there. In the portion of a beam subjected to a positive bending moment, the stirrups, therefore, should be anchored in the bottom of the beam, while in the part of the beam subjected to negative bending moment the stirrups should be inverted and anchored in the upper part.

Summarizing the statements in this paper, we see that:

(1) It is generally uneconomical to design a reinforced-concrete beam stiffer in the middle than over the support.

(2) The middle spans of a continuous beam and fixed beams loaded uniformly may be safely designed for a negative bending moment $\frac{wl^2}{12}$ over the support, and for a positive bending moment of the same magnitude in the middle. The end spans of continuous beams and the adjoining supports may be designed for a positive and negative bending moment $\frac{wl^2}{10}$.

(3) If, to conform with the building laws, a continuous beam is designed for $\frac{wl^2}{8}$ in the center of the span, the top of the support needs to be reinforced with about 6/10 of the steel used in the center.

(4) The compressive stress in concrete at the end of the beam may be, say, 15% greater than the working stress in the middle.

(5) The bond stress in the horizontal steel over the support should receive proper attention.

(6) The stirrups near the support should be inverted, that is, the end in the tensile part of the beam must be anchored.

Yours very truly,
Sanford E. Thompson, M. Am. Soc. C. E.
Newton Highlands, Mass., Dec. 20, 1909.

[Further correspondence on this subject has been received, as follows:—Ed.]

Sir: Referring to Mr. R. E. Spaulding's article on "Moments in Continuous Reinforced-Concrete Beams Under Uniform Loading" and the subsequent discussion of the same, an important feature in connection therewith, I believe, has not been considered.

Among the users of reinforced concrete there appears to be a widespread and growing distrust in the ultimate efficiency of bond. A materialization of this fear is evinced in a recent article by Mr. S. M. Fechheimer in "Concrete Engineering" in which is clearly shown by a series of tests the loss of strength involved in depending on bond in the case of shear members in a reinforced beam.

There are experienced engineers who are willing to go so far as to tell us that the effect of bond should be entirely neglected. Conservative engineering is naturally not prone as yet to accept a radical view of this question, but considering the matter from a rational standpoint it is probably reasonable to assume that the matter should be given due regard in critical cases at least.

It is the purpose of this letter to show the probable

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*This will not hold, however, when a larger percentage of support steel is considered. For instance, if a beam is designed for $M = \frac{wl^2}{8}$ in the middle of the span and *two-thirds* of the steel is placed over the support, then there would be enough steel for about two-thirds of $\frac{wl^2}{8}$ or for approximately $\frac{wl^2}{12}$, and the induced bending moment would call for $-\frac{wl^2}{14}$, as given in the table, so that the resisting moment would be sufficient. Ed. Eng. News.

†A working compression in concrete over the support may be accepted as 15% higher than a normal stress of 650 lbs. per sq. in. in the center of the beam, because at the support the bending moment rapidly decreases and the beam bears against a larger surface.—S. E. T.

effect of partially neglecting bond, upon the aforesaid discussion of beam moments.

Those who regard all reinforced-concrete beams as having a simple end reaction assume that the tensile stress is transferred to the supports directly along the bottom of the beam. Hence the exact position of the stress is known, and may be provided for by a sufficient number of rods running continuously across the span. In other words, there are no "connections" at critical points. Other engineers, with considerable justification, take account of continuous action where it theoretically exists.

In such cases we find the tension fibers on the lower side of the beam at the center changing to the upper portion towards the supports where the negative moments occur.

An attempt is made to follow the curve of the tension stresses by inclining some of the rods at 30° to the horizontal. This method probably meets all practical requirements so far as the beam itself is concerned—especially in the case of unit girder systems. But what becomes of the rods over the supports?

In the majority of cases we find the bars of one beam merely overlapping those of its co-linear beam depending entirely upon bond for the transmission of the tensile stresses over the support.

This may or may not be sufficient in designs including only partially continuous action. However, in the extreme case as mentioned by Mr. H. W. Schlintz (Eng. News, Oct. 28) where the moments $\frac{wl^2}{24}$ at the center and $\frac{wl^2}{12}$ at the supports are used, the vital point is over the support rather than at the center of the beam. Therefore, in conjunction with such bending moment formula it is extremely important to specify absolutely rigid connections of reinforcement.

Moreover, even assuming the reliability of bond, by depending upon this overlapping at critical points it is clear that the transfer of stress will produce tensile strains in the concrete. Since the latter is weak in tension the tendency to produce initial cracks is obvious.

In brief then, for two reasons it is important to specify some form of strictly rigid connection when using highly continuous action formulas.

Very truly yours,

R. E. W. Hagarty.

662 Euclid Ave., Toronto, Canada, Dec. 4, 1909.

A Table of Coefficients for Combined Stresses.

Sir: The following table of coefficients for combined stresses is given as a convenience in computing resultant stresses, that is in all cases which require close figuring of stresses. The coefficients give the ratio of maximum resultant tensile stress to applied tensile stress, for various ratios of shear to tension.

Merriman gives the following formulas for maximum resultant stresses in a piece subjected to both shear and direct stress:

$$S = \pm \sqrt{s^2 + \tfrac{1}{4} t^2}$$

$$T = \tfrac{1}{2} t + \sqrt{s^2 + \tfrac{1}{4} t^2}$$

in which

s = applied shearing stress,
t = applied tensile or compressive stress,
S = maximum resultant shearing stress,
T = maximum resultant tensile or compressive stress.

From the formulas it will be seen that T, the resultant direct stress, is always greater than S, the resultant shear.

For making the table, the applied shearing stress was expressed in terms of the applied direct stress, $s = C_2 t$. Then the formula for resultant tension may be written $T = C_1 t = C_3 s$.

$C_1 = \frac{T}{t}$	$C_2 = \frac{s}{t}$	$C_3 = \frac{T}{s}$
1.1	.33	3.33
1.2	.49	2.45
1.3	.62	2.07
1.4	.75	2.86
1.5	.87	1.72
1.6	.98	1.63
1.7	1.09	1.56
1.8	1.2	1.50
1.9	1.31	1.45
2	1.41	1.42
2.5	1.94	1.29
3	2.45	1.22
4	3.46	1.16
5	4.47	1.12

It will be seen from the table that when $C_2 = 1$, that is when $t = s$, the resultant tension (or compression) is 1.6 times the applied tension (or compression).

The table was computed by the following:

$$C_2 = \sqrt{C_1^2 - C_1}$$

$$C_3 = \frac{C_1}{C_2}$$

which were derived from the original equations. Various values of C_1 were assumed, as given in the first column of the table, and the values of the second column computed by the first, of the above expressions. Then C_3 was computed by dividing the first by the second column.

Chas. E. Evans.

Aurora, Ill., Dec. 13, 1909.

[The allowable shear is usually much lower than the allowable tension or compression. Often the shearing value is taken as half the tensile value. But the equations for combined stress quoted above show that in all cases of combined stresses the resultant shearing stress is greater than half the resultant direct stress, and that it exceeds six-tenths of the resultant direct stress whenever the applied shearing stress is at least seven-eighths as large as the applied tensile stress. These figures indicate that the resultant shear intensity is apt to be the governing factor in many cases, and not the intensity of resultant direct stress. The table given in the letter above does not contain a coefficient for resultant shear, apparently. However, reference to the original formulas for S and T shows that $S = T - \frac{1}{2} t$. Since T is $C_1 t$, we obtain $S = (C_1 - \frac{1}{2}) t$. In other words, if from the tabular values of C_1 we deduct 0.5, we get the ratio of resultant shear to applied tension. Thus, in a shaft where the extreme-fiber tension due to bending is 8,000 lbs. per sq. in., and the torsional shear 5,500 lbs. per sq. in., we have

$$C_3 = s/t = 5{,}500 \div 8{,}000 = 0.69,$$

and from the table

$$C_1 = 1.35.$$

$$\text{Then } C_1 - \tfrac{1}{2} = 0.85,$$

and the resultant shearing stress is

$$S = 0.85 \times 8{,}000 = 6{,}800 \text{ lbs. per sq. in.}$$

As the resultant tension is $1.35 \times 8{,}000 = 10{,}800$ lbs. per sq. in., the ratio of shear to tension is $6{,}800 \div 10{,}800 = 0.63$. If the allowed limit for shear is 0.6 or less of the allowed tensile stress, shear would govern in this case. On the other hand, if the allowable shear is 0.8 the allowable tensile stress, then the applied shear must be at least twice the applied tensile stress before shear will govern. In any specific case the matter can be examined by finding the resultant shear, as above indicated.—Ed.]

Methods of Improving Published Cost Data.

Sir: It is well to call attention to the great variation in the treatment of cost data by engineers and the unreliability of a great deal that is published on the subject, as you have done in your issue of Dec. 23, but it seems to me that you should go further and call for suggestions on the best way to remedy these faults or even lay down rules to be followed in collecting and working up cost data.

Among the careful articles on the subject, the chief difficulty seems to be in the method used in figuring the material obtained on the job. A number of principles regarding the method of collecting and recording the cost of items in structural work could be stated at the outset and these might be added to, from time to time, by discussion in your columns, the ultimate idea being to get as close as possible to a standard method. For example, temperature and weather conditions; the location and purpose of the structure; whether built by the public, a corporation, or individual; the closeness and quality of inspection; a statement of labor conditions and wages; and an account of the method of collecting the data are all obviously needed to make any collection of cost data useful to other people; and yet some of them are often omitted.

It should be a rule never to state labor cost in terms of dollars and cents, but always in terms of hours of labor of a certain class. A great deal of otherwise valuable data is rendered almost worthless for reference by the fact that the labor cost is figured out as so many dollars. An engineer in any other section of the country than that near the work in question would either have to go over all the data or else depend on his good judgment to take account of the difference in wages, etc., in order to use it at all. For similar reasons the cost of labor and material should never be combined. Stating the labor cost in terms of time and the material cost in terms of quantity would make the items valuable in spite of the rise and fall in wages and prices. Cost data on this basis might be confusing at first, but this method is, in my opinion, the only way to collect data for reference. It is easy enough to reduce such items to the money value of the time and place under consideration.

The matter might be further simplified, if we assume that the difference in wages in any given time and place represents the economic worth of the individual to that locality, and reduce all labor cost to a basis of the cost of "common labor." A foreman who receives $4 per day while the laborers under him receive $2 has double weight in determining the cost and if the gang is properly organized is doubly valuable as a worker. Why not figure his time as two days of laborers' time?

As an illustration, suppose a gang of 12 laborers at $2, two laborers at $2.50, an engineman at $4, and a foreman at $3 are able to place 60 yds. of concrete in ten hours. As is customarily figured, the labor cost of mixing and placing the concrete is $0.633 per cu. yd. From the point of view of usefulness to others, wouldn't it be much more sensible to state the cost as 3⅛ hours of labor at 20 cts? This value could be obtained by adding together 1 for each of the laborers, 1¼ for each of the two more experienced laborers, 2 for the engineman, and 2½ for the foreman. This makes the gang equivalent to 19 laborers or 190 hours or 3⅛ hours per cubic yard of concrete.

The same result could be obtained by dividing the cost price of $0.633 by $0.20, the cost per hour of laborers. This method affords a good means for comparison of the labor costs on various jobs in different parts of the country and a ready means of reducing other data to a common standard. The number and class of men in each gang should be stated, and the number of hours of common labor per unit of work will then be a proper criterion of the efficiency and organization of the gang.

The above is merely a suggestion toward more flexibility, and usefulness in published cost data. Engineering News would do a great good if it could succeed in bringing some uniformity out of the chaos that exists at present in the preparation of published cost data. The chief trouble seems to be in not knowing exactly what is wanted.

Yours sincerely,

Clark Rogers Mandigo,
Civil Engr. and Supt. Const.

Fort Riley, Kan., Dec. 31, 1909.

Costs of Tunnel Work on the Los Angeles Aqueduct.

Sir: I have had during the past few days my first opportunity of seeing the article in your issue of Nov. 18, describing the methods used and giving in detail the cost of Tunnel No. 7 of the Los Angeles Aqueduct, and wish to congratulate you and the writer on the very complete information given in it. I do not know of any account published anywhere which gives so much and so complete information of just the kind one wants to know in regard to rock tunnel construction, without which bare records of progress and statements of low cost are almost valueless.

There is only one item which is not quite clear. The record covers a 15-day period, but reference is made to the fact that 90 ft. was driven in 15 8-hr. shifts. There was probably more than one shift worked per day and possibly this refers to drilling shifts. It would be interesting to have this made quite clear and to know the number of 8-hr. shifts worked in fifteen days.

Yours very truly, F. Lavis.

50 Church St., New York, Dec. 31, 1909.

[As stated in the 9th paragraph of Mr. Richards' article, the drillers worked in a single day shift, while the muckers worked at night. In 15 days there were 15 drilling shifts. A different working schedule prevailed in the Red Rock Tunnel, as recorded in the article immediately following the one to which Mr. Lavis refers.

The two articles afford numerous instructive contrasts, besides in arrangement of shifts. Thus, the Red Rock Tunnel was in very soft rock, but did not require timbering, while Tunnel 7 was in hard granite but had to be timbered throughout the 15-day period. The costs recorded in the two works are therefore not at all comparable, and it is even a noteworthy agreement that one cost only twice as much as the other per foot of length.—Ed.]

Notes and Queries.

A correspondent (Mr. A. D. Payson, 1287 North Wells St., Chicago) writes that he would like to see in these columns some expressions of opinion and experience on the relative merits of pedometers, which register paced distance directly, and of passometers, which register the number of steps from which the distance may be computed. The actual degree of approximation of both is desired as found by considerable experience, and the reliability of the "self adjusting" action of pedometers compared with the use of an average pace in passometer computations, particularly in connection with preliminary surveying or rapid mapping.

THE LIBRARY OF CONGRESS at Washington, D. C., is increasing at a rapid rate and is now probably the third largest collection of books in the world, passed only by the British Museum in London and the Bibliotheque Nationale at Paris. During the fiscal year ending July 1, 1909, 167,677 volumes were added, making the total number in the library 1,702,685. The collection of maps and charts now numbers 111,343, the music collection comprises 501,293 volumes and pieces, and the statistics of the prints division show 303,036 pieces.

The Failure of a Concrete Dam at Dansville, N. Y.

On the morning of Dec. 5, 1909, a small concrete dam across the Canaseraga Creek at the plant of the George Sweet Manufacturing Co. failed under a height of water just below the minimum overtopping flow for which the dam was designed. A portion of the dam was bodily separated from the remainder and carried intact about 50 ft. downstream and the part remaining was undermined at one end so that about half of it was dropped some feet. Since the flow in the stream at the time was not extraordinary the study of the dam and its failure should prove profitable.

During the past year the George Sweet Manufacturing Co. has found it necessary to enlarge its plant on the shore of the creek, and in the course of this enlargement decided to build a new dam about 1,000 ft. downstream from its old dam to increase the water storage. The creek is at the head of the Genesee Valley and is subject to very sudden and very heavy floods, so that the construction had to be carried out as rapidly as possible to take advantage of the low water. For that reason, and partly on account of the nature of the foundation as described hereinafter, a buttressed-slab reinforced-concrete dam was decided upon.

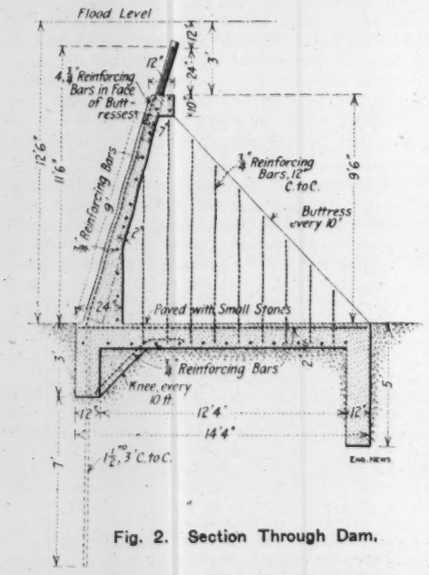

Fig. 2. Section Through Dam.

Investigation of the river bottom showed that, except for a distance some 40 ft. out from the north bank of the river, no rock bottom was to be found within a reasonable depth, but that a good solid gravel was on the river bottom for its whole width. This gravel was thought to be dense enough and solid enough on which to found the wide bottom slab of the skeleton dam. In the construction it was found to be so stiff that the trench for the heel wall, 5 ft. deep, could be dug and held without sheathing until the concrete was placed. The design decided upon and afterward built is shown in Figs. 1 and 2. As will be noticed there, it is of the buttressed reinforced-concrete type, with a slightly sloping deck face. The bottom slab consists of a horizontal slab 12 ins. deep, with a toe cut-off wall 3 ft. deep and a heel cut-off wall 5 ft. deep. The toe wall is connected by a fillet at each buttress—that is, at every 10 ft.—and in addition is reinforced by steel rods through the fillet. In the excavation made for this toe cut-off wall, iron rods 1½ ins. in diameter were driven 10 ft. into the solid gravel and the concrete laid up around them, thus anchoring the toe to the gravel after a fashion. The heel wall was neither reinforced nor anchored. The buttresses, as noted before, were spaced 10 ft. c. to c., and were reinforced with vertical rods. The front slab was reinforced by vertical rods at each buttress and by horizontal rods for its whole distance. The stress sheets for the dam as designed give a uniform pressure on the base of 535 lbs. per sq. ft., under an assumption of uniform transmission of the load over the whole floor slab. On account of the lack of reinforcement at the heel it is quite probable that this pressure was measurably increased at that point, because of the inability of the heel wall to transmit the maximum pressure along the whole 10 ft. between buttresses.

In connection with the dam proper there were a 21-ft. 6-in. sluiceway on the north side and a 25-ft. earth embankment on the south side. This north sluiceway was a gate slung between two heavy buttresses, shown on Fig. 2, the northern one of which was anchored by rods into the rock foundation. The southern buttress wall of the sluiceway reached to bed-rock but was not anchored there. The 5-ft. toe walls up to the third buttress south of the sluiceway reached to bed-rock, but south of that point the rock sloped off below the 5-ft. depth. The earth embankment on the south end was built of non-puddled clay. From the south end of the south buttress of the dam, a core wall, marked on Fig. 2 "Old Core Wall," extended at a sharp slope into the embankment. This was afterward replaced, as hereinafter described, by another core wall at a greater slope, marked "New Core Wall" on the drawing.

The bottom slab of the dam was paved with a layer of stones, rammed solid into the wet concrete before it had set. These stones were to take the batter of the water which might overtop the dam. Into this slab were cast weep holes, as shown in Fig. 2. The top of the dam was arranged to take two sets of flashboards so as to increase its height to 11 ft. 6 ins.

When the water was first let in behind the dam, the embankment at the south end washed out around the original core wall, and the new core wall shown was then put in and the earth replaced. Water was then again let in behind the dam, and, rising slowly, reached on Dec. 4 a height of 10 ft. above the floor slab. At this time a crack was noticed through the face of the wall at a distance about 36 ft. from the south buttress of the sluice. This crack was not sufficient to allow spouting. Some observers also claim that water was seen spouting from the weep holes in the floor slab to the height of

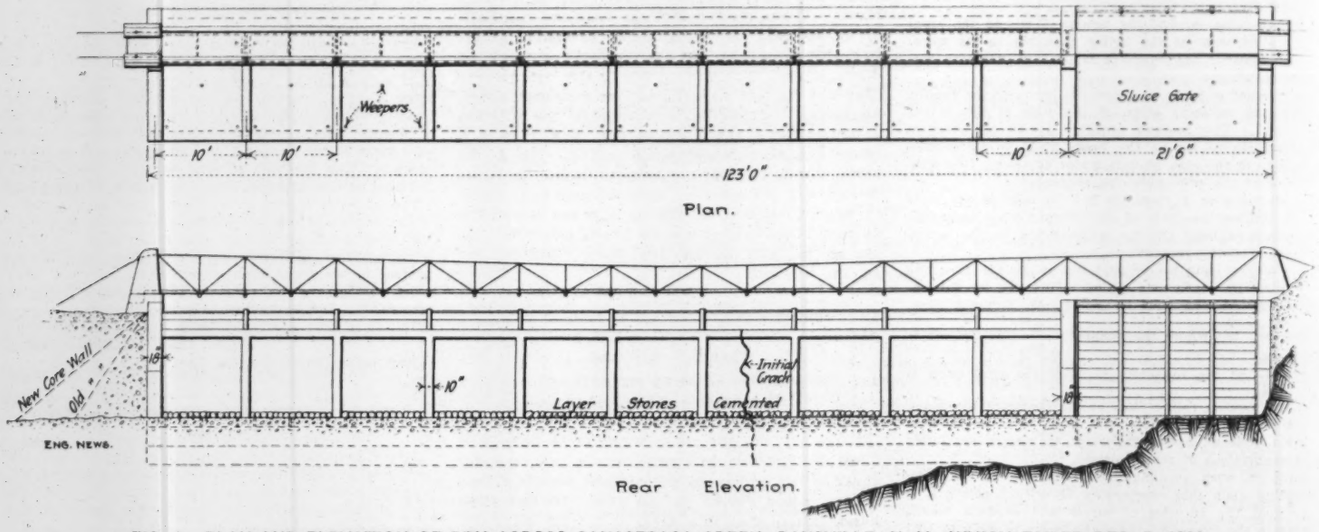

FIG. 1. PLAN AND ELEVATION OF DAM ACROSS CANASERAGA CREEK, DANSVILLE, N. Y., WHICH FAILED DEC. 5, 1909.

Fig. 3. Near View of Original Break in the Dansville Dam, Showing Termination of Reinforcing Rods at this Section.

some inches, but this is denied by the engineers for the company, who say that at no time did the gravel below the dam show signs of under percolation. However, the next morning, Dec. 5—no eye-witnesses being present—the dam broke at the point of this crack. The wall, from the crack to and including the south buttress of the sluice, was picked up bodily and carried about 50 ft. downstream, landing in the location shown in the half-tone in Fig. 4. The rushing water undermined the dam at the break, and swung the remaining portion slightly downstream and dropped it in the hole formed from the under-wash, cracking the wall at a point about 40 ft. south of the original crack. This constituted the entire damage.

The design and construction of the dam were under the direction of the company, who let the contract to a local man and appointed its own inspector for the work. There were no specifications for the construction and the degree of observance of the drawings is in doubt. In the investigation of the dam after the collapse it was found that the construction had been carried out in the poorest manner possible. The concrete had not been properly mixed nor of the right proportions, so that large lumps of almost pure cement and of hardly cemented sand and gravel were found in places. More important than this, it was found that the line of the original break was also a line of a joint in the horizontal rods in the deck face of the dam, so that instead of the rods over-lapping and forming a continuous tensile reinforcement across the dam, there were one or more vertical lines of weakness in the face where the only resistance against bending was provided by the concrete slab there. The line of the secondary break was another of these same reinforcement joints which naturally gave way under the strain following the original break. Fig. 3 shows the break at the original line of collapse with the dead ends of all the reinforcing rods showing clean at the break. Not one of the rods here or at the other break were either tied to its continuation or overlapped. In view of the poor concrete and the failure to observe ordinary precautions on the face slab rods, it is possible that the remainder of the dam was not built to drawings.

The under-wash after the break was so great that all the gravel bottom in the vicinity of the section which was carried away was torn out, thus destroying any evidences as to the point of fracture of the cut-off walls. On the portion of the dam washed downstream the cut-off walls have both broken from the floor slab, but it is impossible to ascertain whether this break occurred after the piece was torn from the rest of the dam or whether the upper portion sheared off from the walls, which were afterward washed out of their excavation by the rush of water. On account of its lack of reinforcement it is quite probable that the heel wall sheared off when the dam first started to move, but the steel in the toe wall may have helped it to remain intact until the shock of being washed downstream broke it off. Elsewhere along the dam the mud which had been banked up on the upstream face as a partial protection against possible under-wash is still in place.

FIG. 4. VIEW LOOKING UPSTREAM AFTER FAILURE OF DAM.

German Industrial Methods.

A comparison of the British and German electrical industries, much in the nature of a report on examinations of German factories, was presented in a paper before the British Institution of Electrical Engineers on Dec. 9, 1909, by Messrs. L. J. Lepine and A. R. Stelling. From an abstract of this paper, in the London "Electrician" of Dec. 10, a few points of interest have been further abstracted.

FINANCE.—During the dull period of 1901-1904 the numerous manufacturing concerns, with their associated trading and investing companies, were amalgamated* into four large corporations: the Allgemeine Elektrizitäts Gesellschaft, the Siemens-Schuckert, the Felton-Guilleaume-Lahmeyer, and the Bergmann companies. These concerns have a heavy capitalization allowing them to meet easily any sudden demands caused by the advance of practice, such as the special plants for construction of turbo-machinery. The order for work on the Victoria Falls power scheme resulted from their ability to contribute a large part of the necessary capital and to erect a plant for building special machinery.

In the improvement of the trade the material assistance afforded by financial syndicates has been of such value that during 1908 a further step in electrical finance was taken. This was the establishing of two electro-trustee banks (Elektrotreuhandbanken) under the control of the large German electrical companies. The principal object of these "trustee" banks is to advance to large consumers in the electrical industry the money they may require for beginning and carrying out costly works. The capital needed for the loans is to be obtained by mortgage bonds, secured in the first place by the stocks held by the bank, and in the second place by the share capital of the latter which is to be invested in trustee stocks.

The various financial institutions, by providing the manufacturers with working capital and credit, enable them to compete successfully in all markets and render each firm independent of the others where capital is required. But should the activity of the banks be reduced from any cause, or should foreign and local competition become so keen as to cause repeated price cutting, then the great disadvantages of over-capitalization will make themselves felt, and the large firms will be driven to the formation of a huge trust with their associated banks as a nucleus, their existing intimate relationship rendering the accomplishment of this all the more easy.

ORGANIZATION.—The home trade is carried by the main and local offices, but foreign trade is entrusted to subsidiary concerns having share capitals varying from $500,000 to $3,000,000. These companies are all provided with branch offices, which are controlled by the subsidiary company financially, but which deal directly with the factory in technical matters. Again, complete staffs are maintained in the countries outside Europe where many competitive English firms are content to employ agents; often enough the latter have but the faintest ideas on the subject of electricity and its application. Thus, German firms are always in a position to treat inquiries from all countries with the utmost consideration, and to furnish very complete tenders which are sure to meet customers' requirements.

In the offices a bureaucratic rule obtains. The keynote of the whole system is that each man's work is specialized, and each branch of manufacture is an organization in itself with its own drawing office, etc. A publicity department run by university graduates in literature is responsible for the catalogue and journalistic work. Besides this department and the estimating offices for the home trade, each firm has a foreign department which deals exclusively with inquiries from abroad, and which is in correspondence with the branch offices (outside Germany) and the subsidiary companies. All these various departments are subject to the "Verwaltung," or central authority, which also deals directly with the employees.

All members of the technical staff go through one of the drawing offices, and are usually bound under a contract which is all in favor of the employer. Salaries are kept low; after leaving college the engineer commencing work at about 24 years of age receives $31.25 a month. This increases with machine-like regularity until it reaches the maximum wage for the department. Holidays of more than three days, either from head office or from a branch office, must be granted by the central authority.

SHOP METHODS.—In the works of the four large concerns named order and cleanliness prevails, though the ventilation needs improvement. All the machine tools have excellent protective devices on account of strict factory regulations and inspection. With plenty of available capital, large numbers of special machines have been installed. While automatic machinery and turret lathes are largely used, jig work does not receive the same attention. The majority of machine tools are of American make. Practically all iron and steel castings are purchased outside but the firms have their own brass and aluminum foundries.

The most striking feature in the works systems consists of an 18-hour workday, of two shifts of nine hours each—from 7 a. m. to 4.15 p. m. and from 4.45 p. m. to 2 a. m., with a quarter-hour interval in the middle of each shift. The rate of wages is low, even to the skilled mechanic. The average wage is about $8 a week and winders on high-tension generators earn about $9 as a maximum. Female labor is very largely employed and women can earn higher wages than those similarly employed in England and they possess special privileges under thoughtful legislation. Piece work is the rule in erecting—one man is given finished units, a number of assistants and a fixed sum for erection out of which all wages are paid.

In individual workmanship, the German firms do not compare with the English. Careless and untidy taping and stringing of coils, lack of finish and the "deep scratch and high polish" appearance are often noticeable.

SCIENTIFIC EXPERIMENTAL WORK IN FACTORIES.—It is the thorough system of scientific investigation and experimental work that has enabled German firms to attain otherwise inaccessible eminence in certain branches. Perhaps the best known example is the Zossen experimental high-speed railway. Another example is an important series of investigations carried out on mining motors. Less well known are the important tests of the effect of heat and age on insulating materials carried out by one of the large firms and of which no results have yet been made public.

As an example of the application of scientific methods to works practice may be cited the dynamic balancing of all rotors and armatures of high or extra high peripheral speeds, as well as of all large-size rotating parts while in the building-up stage. Of course, capital is required for these experiments, but as the German engineers are well trained theoretically, and as the German firms are willing to employ them, the capital is rarely expended without some tangible profit.

*An article in Engineering News, Feb. 4, 1909, described the evolution of the present German concerns.

CONTRACTS FOR THREE DIKES at various points on the margin of the Ashokan Reservoir, now under construction by the Board of Water Supply of New York City, were recently awarded to MacArthur Brothers Co., New York City. These are known as the Hurley dikes, and are quite separate from the Olive Bridge Dam, which is the main structure forming the reservoir. The following information regarding the character of the dikes has been kindly sent to us by the contractor named above:

The longest of the dikes is approximately 3,500 ft. and will be 60 ft. high at the highest point. The other two dikes are each about 3,000 ft. long and from 20 to 40 ft. high. The dikes will be built with a concrete core-wall with an earth fill on each side, which earth fill is covered with riprap on its outer slopes. The core-wall will extend from bed rock to within about 10 ft. of the top of the dike.

In the case of the Glenford dike it will be necessary to carry the core-wall down through 30 ft. of earth in order that it may be founded on bed rock. The earth embankments are constructed by depositing the earth in horizontal layers not exceeding 6 ins. and rolled with heavy steam rollers, thus making an embankment which is absolutely impervious to the flow of water.

The quantities involved in this construction work will include 45,000 cu. yds. concrete and 900,000 cu. yds. embankment, and will cost about $1,000,000.

The contractors expect to use on this job half a dozen steam shovels with their complement of locomotives and cars and the necessary plant for handling concrete and quarrying the riprap. A force of about 500 men will be placed on this work, which will take about three years to complete.

A summary of the bids received for these dikes was published in our Construction News Supplement for Dec. 9, 1909.

A Centrifugal Sewage-Sludge Drier.

Centrifugal sewage-sludge driers have been put in use by several German cities, including Hanover and Frankfort-on-the-Main. Although there seems to be some question as to the economic feasibility of these driers, at least under American conditions, they seem to be of sufficient interest to warrant reprinting the following descriptive matter from London "Engineering" for Oct. 15:

The machine consists of a dryer, which separates the water from the solid matter by means of a centrifugal action. In operation, two of the machines are installed in conjunction with a sludge-tank, mounted overhead, and provided with mechanically-operated stirrers, to ensure that the material supplied to the machines shall be as uniform in consistency as possible. This sludge-tank is filled from a settling-tank, and it is advisable to so proportion the plant that the sludge may be treated when it is not more than two or three days old, in order that putrefaction may not set in. The settling tank should be filled through a grating, made with about 5/16-in. bars, in order to trap any large objects, which might injure the machine if they were introduced into it. The mass which is ejected after drying contains from 50 to 70% of water, as compared with 90 to 95% in the original sludge, and the volume is reduced by about 80 to 85%.

The machine consists essentially of a drum, with a vertical hollow shaft, revolving at a high speed. This drum is fitted with six radial chambers arranged in a star form, each chamber being divided into two sections by a sieve down the center; one of these sections has parallel sides, and is fitted with valves at the inner and outer ends, these valves being operated by oil under pressure; while the other is of a radial form and contains a drain-pipe communicating with an annular canal situated below the drum.

The method of operation is, briefly, as follows: With the drum revolving, the inner valves open and the outer ones closed, the feed-valve from the tank above is opened and sludge is introduced into the parallel sections of the chambers by way of the hollow shaft and the inner valve-ports. The solid matter present is then immediately carried, by centrifugal force, to the outer part of the chambers, the lighter water being forced back, and thus separated from the solids; this separated water passes off through the sieves into the drain-pipes and out of the machine by way of the annular canal. As the process is continuous, the parallel part of the chambers finally becomes filled with a dried mass, the whole of the separated water passing off by the overflow. The inner valves are now closed, and the outer ones opened, when the dried mass is projected outwards against the casing of the machine, striking which, it is broken into small pieces and falls, by way of a funnel, into a receptacle or conveyor below. In the act of being projected outwards, the dried slude slides past the sieves, and thus frees them from small adhering particles which might tend to choke up the meshes and impede the free passage of the water.

Referring now to the machine in more detail, it will be seen from Fig. 1 that it consists of an external shell built up of plate and angles, inside of which the drum, with its radial chambers A, revolves. The hollow shaft B is carried in bearings at the top and bottom, the bottom one being a footstep bearing working under oil pressure, the oil service which is used in connection with the operation of the sludge-valves being employed for this purpose. The oil is carried to the operating cylinders through holes in the lower portion of the vertical shaft as shown, entering through the gland at C and debouching at D. The shaft and drum are driven by the belt pulley E, while the valve mechanism for the cylinders which control the sludge-valves is operated from the pulley F, which is independently driven from the machine counter-shaft by a second belt—this is clearly shown in Fig. 2, which illustrates an installation of these machines. The wiper-shaft carrying this second pulley is fitted with a small crank, which controls the oil-valve mechanism of the cylinders by means of the link and bell-crank lever shown, and the sleeve G; this sleeve is connected with the mechanism by means of six small vertical links. The sludge-valve operating cylinders are clearly shown in the lower sectional plan, and it will be understood that the cylinders, with the whole of their mechanism, revolve with the drum, the only connection between them and the fixed part of the machine being through the sleeve G and the oil inlet and outlet C and D. The pulley fixed on the upper part of the casing, and driven by a belt from the wiper-shaft, is used, by means of the worm and worm-wheel shown, to operate the auxiliary drum H. This drum is suspended from the ends of the two weighted rocking levers, and is normally in the position shown in Fig. 1; immediately before the opening of the outer valves for the ejection of the dried mass, however, it is raised, so that the mass may be free to impinge against the outer shell of the machine and then drop into the funnel below.

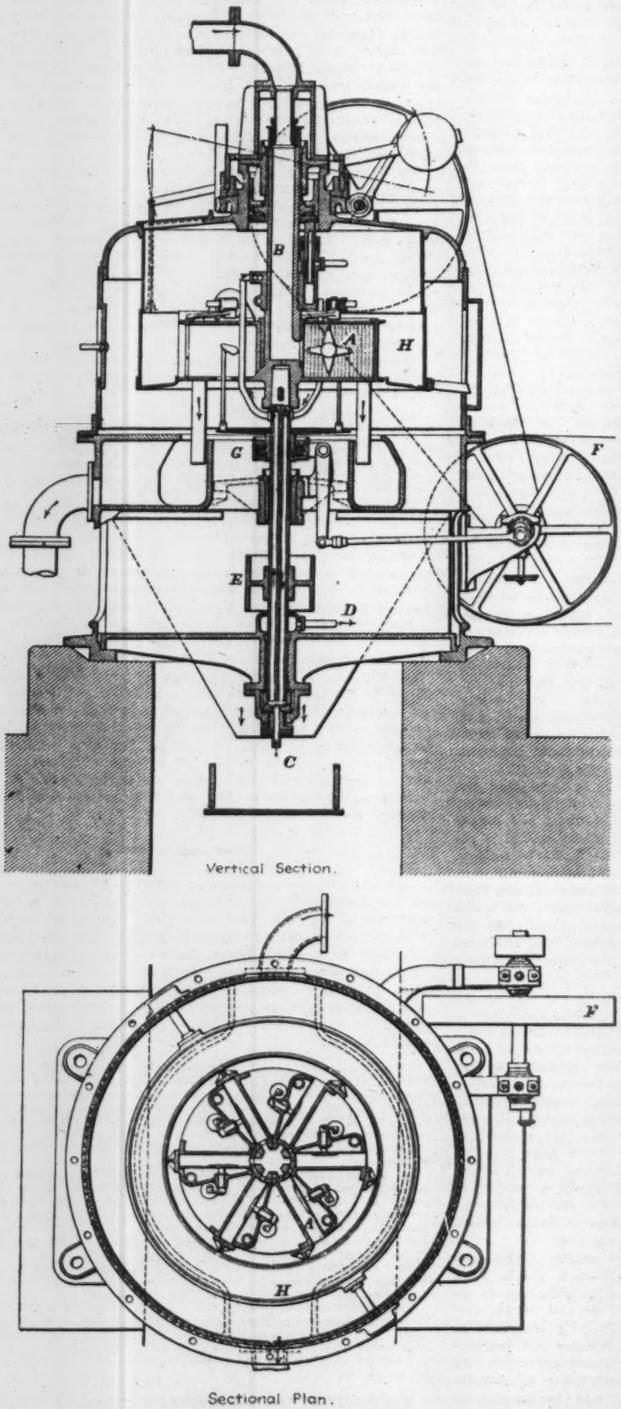

FIG. 1. SECTIONAL PLAN AND CROSS-SECTION OF CENTRIFUGAL SEWAGE SLUDGE DRIER.

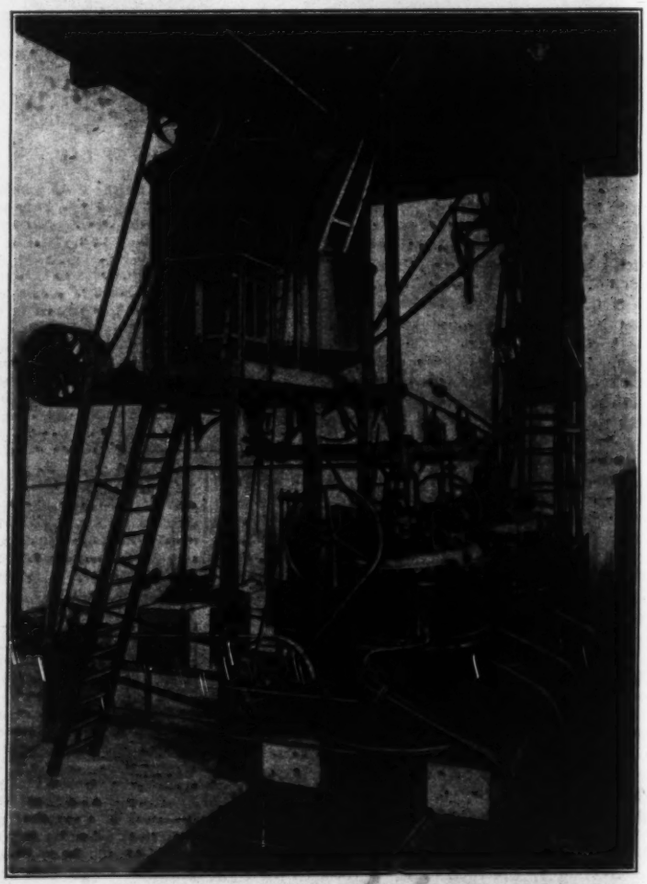

FIG. 2. INSTALLATION OF CENTRIFUGAL SEWAGE-SLUDGE DRIER AT HARBURG-ON-ELBE, GERMANY.

The drum is raised by the swinging of the rocking levers, small projections on these being engaged by wipers on the rotating worm-wheel at the necessary moment. The object of this auxiliary drum is to trap any water which may be shot out radially from the revolving drum owing to its high speed, and it is in communication with the main outlet-water pipes from the apparatus by means of drain-pipes and pockets in the main casing. When the auxiliary drum is raised, the pockets are covered on the inside by small plates attached to the drain-pipes, in order to prevent portions of the dried mass lodging in them and choking them up. These plates are not shown in the illustration.

An installation of two of these machines, which is illustrated in Fig. 2, has been in operation at the town of Harburg-on-Elbe since the end of 1907. The sewage works consist of four vertical settling-basins, from which the deposited sludge is drawn by suction into a sludge-tank, and thus forced by compressed air into the tank above the machines. The residual water, after separation, is led by an earthenware pipe into the main sewer, and so back to the settling basins. No unpleasant smell is experienced with the apparatus. The amount of sludge treated per day is from 530 to 700 cu. ft., the amount of water contained in it varying from 88 to 95%. This amount of sludge can be dealt with by one machine working seven or eight hours, and the total energy required to drive the plant during this period is 100 KW-hrs. The plant is electrically driven, current being purchased from the municipality at about 0.6d. per KW.-hr., so that the cost of power for drying 100 cu. ft. of sludge amounts to from 8½d. to 1s. [16 to 25 cts.] It is found that one man only is required to attend to the sludge-drying plant proper. Another installation, on a larger scale, has recently been put into operation for the town of Hanover. In this case four machines are in operation, with twelve settling-basins, which deposit about 2,800 cu. ft. of sludge per day. The dried mass is carted away and used for filling up low-lying land. The whole settling and drying plant is driven by a 75-HP. suction-gas installation.

A further plant is in course of erection at Frankfort-on-the-Main. The installation consists of six machines, each driven by a separate electric-motor; three mud reservoirs, each of 30 cu. m. [40 cu. yds.] capacity, and each furnished with a churning apparatus; together with conveyors and elevators and duplicate oil-pumps, space being left for two further machines. The installation is intended to work day and night, and deal with from 200 to 250 cu. mt. [260 to 325 cu. yds.] in the 24 hours, and will be put into commission this autumn. It is proposed to burn the dried mass ejected from the machines in a destructor in conjunction with town refuse, which, of course, contains a considerable amount of ashes and other combustible matter.

This device is known as the Schaefer-Ter Meer's centrifugal drying machine and is made by the Hannoverische Maschinenbau Actien-Gesellschaft, Hanover, Germany.

We supplement the foregoing description by a brief mention of and comment on the machine and its probable place in sludge treatment taken from "Principles of Sewage Treatment," an English translation of a German work by Dr. Dunbar, Director of the Hamburg (Germany) State Hygienic Institute. The preface to the German edition was dated July 26, 1907, which would indicate that the machine was tried at Frankfort several years earlier than would be inferred from the article published by "Engineering." Writing of sludge from settling and chemical precipitating tanks Dr. Dunbar says:

> Numerous experiments have been undertaken with the object of converting this sludge into a solid transportable mass. At Frankfort this problem has again been recently attacked in a thorough manner. By means of a centrifugal machine they have succeeded in obtaining in a few minutes a solid mass containing about 70% of moisture. Just as in pressing so in centrifuging, the addition of lime (15%) proved to be advantageous. It is intended to mold the centrifugalized sludge into briquettes by means of brick presses, to allow the briquettes to dry in the air like bricks, and then either to grind them up for manure or to burn them for heating purposes and the production of gas.
>
> The question of the treatment and disposal or utilization of the residues at town sewage works remains still under discussion. Each of the many processes recommended raises hopes of financial gain. The Frankfort experiments have demonstrated that quick drying of the sludge and its conversion into an unobjectionable product is only possible as yet by means of a very complicated and expensive process.
>
> Wherever it is possible to obtain a sufficient area of suitable land in the neighborhood of the sewage works, the cheapest method of disposal will be to conduct the sludge into furrows and cover it with soil.

A 20-IN. HIGH-PRESSURE WATER MAIN BURST at Boston, Mass., Jan. 3, and flooded the district bounded by Tremont, Eliot, Lagrange and Washington streets and, as far east as Harrison Ave., between Kneeland and Beach streets. The flood lasted for more than an hour and reports state that in Washington St., between Lagrange and Eliot, the water was more than 3 feet deep. The flooded streets were impassable except to horse-drawn vehicles. Considerable damage was done to business houses and theatres and a quantity of water poured into the Tremont St. subway through a ventilator grating. The total property loss is estimated at $175,000.

AN EXPLOSION OF DYNAMITE on the Oregon Trunk Line near Dalles, Ore., Jan. 8, killed eight laborers.

THE SUBSIDENCE OF A DISUSED MINE at Raibl, Carinthia, Austria, Jan. 8, engulfed a small hospital building. Seven persons were killed.

BRIDGE SHOP FIRE.—Milwaukee plant of the American Bridge Co., at Milwaukee, Wis., was destroyed by fire on Jan. 4. The plant was situated at the foot of 18th St. in the Menomonee Valley. Four firemen were killed by the falling of a portion of the roof. The loss is estimated at $100,000.

AT YOUNGSTOWN, OHIO, Jan. 7, the machine shop, pipe shop and electrical shop in the works of the Republic Iron & Steel Co. were destroyed by fire. Other buildings of the plant were damaged.

THE COLLISION OF A WORK TRAIN and an extra freight on the Chicago, Milwaukee & St. Paul Ry. between Gretna and Roscoe, S. D., Jan. 4, resulted in the immediate death of three workmen and the fatal injury of three others.

NO FATAL ACCIDENTS TO PASSENGERS have occurred on the Delaware, Lackawanna & Western R. R. since Jan. 1, 1900, according to an announcement recently issued by the officials of that road. During this period of ten years, the Lackawanna has transported 193,787,224 passengers with an average length of trip of 19.91 miles for each passenger. The number of train-miles for passenger trains was 65,340,908.

A SWITCH ENGINE WRECKED A PULLMAN CAR Jan. 10 in the yards of the Texas & Pacific Ry. at Shreveport, La. Two passengers, a porter and three trainmen were killed. Twenty other persons were injured. The wrecked car was on the rear of passenger train No. 53, westbound, which was backing through the yards from the passenger station. The switch engine, running at high speed to clear the track for another train, then due, crashed into the rear of No. 53. The accident occurred at about midnight.

AN EXPLOSION IN A COAL MINE at Nottingham, Pa., Jan. 11, killed six miners and injured four others. The victims of the accident were making repairs in one of the lower levels of the mine and the explosion is ascribed to the ignition of gas by a carelessly opened safety lamp. The mine is owned by the Lehigh & Wilkesbarre Coal Co.

A HEIGHT OF 3,600 FT. was reached by Hubert Latham, at Mourmelon, France, on Jan. 7, driving an Antoinette monoplane. The best previous record (unofficial) was 1,600 ft. and was made by Orville Wright at Potsdam, Germany, on Oct. 2, 1909.

The Los Angeles aviation meet, announcement of which was noted in Engineering News, Dec. 30, 1909, opened on Jan. 10. The first day saw only exhibition flights of aeroplanes and dirigible balloons, but the spectators were given a few thrills by the spectacular flights of the French aviator, Louis Paulhan. On the second day, Glenn H. Curtiss established a speed record, for aeroplanes carrying a passenger, of 55 mi. per hr., over a short course.

A TRAIN WITH A TOTAL WEIGHT OF 9,120 TONS was recently hauled by a single locomotive, over the Virginian Ry., from Victoria, Va., to Sewells Point, near Norfolk. As described in the New York "Tribune" of Jan. 9, the train consisted of 120 coal gondola cars hauled by a Mallet locomotive. Each car carried 55 tons of coal, and the run of 125 miles was made in 8 h. 11 min. actual running time, or 9 h. 5 min. including stops. The Mallet locomotive hauling the train has 300,000 lbs. on the drivers and 18,000 lbs. on the leading truck. The tender weighs 163,000 lbs. loaded, and has a capacity of 9,500 gals. of water and 15 tons of coal. The railway on which this run was made is described on pages 34-36 of this issue, but the details of this record-breaking train were received too late for inclusion in that article.

THE QUEBEC BRIDGE DEBRIS.—The contract to remove the debris of the fallen portion of the Quebec bridge on the south shore has been awarded to Capt. Charles Koenig. The work has already been begun and some thirty men are employed. According to the contract the first piece has to be cleared by the 1st of May next, and when the work progresses there will be as many as one hundred or more employed. Numerous offers have been received for the purchase of the debris, but so far none have been accepted and the damaged steel is being piled up on the shore. The difficulty of the work is easily estimated and it is considerably increased owing to the excessive cold, the ice and snow. The work of severing the tangled steel is being effected by means of a mechanical cutter.—(Montreal "Gazette," Jan. 5.)

The most modern machinery will be used to cut apart and remove the different sections of the wrecked superstructure, special chemical apparatus being employed which will smelt apart the steel bars that collapsed.—(Montreal "Herald," Jan. 4.)

AN ORDER REQUIRING HIGH-POWER HEADlights has been issued by the Indiana State Railway Commission [W. J. Woods, J. C. McClure, F. E. Payne, Commissioners] under date of Jan. 7, 1910. All locomotives except those used in switching are required to be equipped with headlights of not less than 1,500 c. p. Every railway is required to equip at least one-third of its engines by July, 1910, two-thirds by January, 1911, and all by July, 1911. An interesting introduction to this order will be found in Eng. News, June 10, 1909, p. 643, under the title "A Hearing on Electric Headlights Before the Indiana State Railway Commission." The evidence taken at that time was adverse to high-power headlights, but since that time the commission's inquiries have convinced it that high-power lights are distinctly desirable. A law requiring high-power headlights in the state of Washington led to tests reported by Mr. H. L. Gray in our issue of Dec. 30, 1909, p. 716, "A Comparative Test of Electric and Acetylene Headlights on Locomotives," which the Washington commission interpreted as favorable to electric headlights.

Personals.

Mr. E. N. Chisholm, of Columbia, S. C., has become Superintendent and Manager of the water system of Knoxville, Tenn.

Mr. John M. Hackett has been elected City Engineer of Dunkirk, N. Y., for a term of two years, to succeed Mr. James P. Morrissey.

Dr. E. J. Lederle has been appointed Health Commissioner of the City of New York. Dr. Lederle is a member of the firm of Lederle & Provost, sanitary and hydraulic engineers.

Mr. Alfred P. Prendergast, Assistant Master Mechanic of the Baltimore & Ohio R. R., at the Mount Clare shops, Baltimore, Md., has been made Master Mechanic to succeed Mr. Chas. T. Turner, retired.

Mr. A. F. Potter, formerly a member of the Forest Service in charge of the regulation of grazing on forest reserves in the West, has been appointed Acting Chief Forester to succeed Mr. Gifford Pinchot, dismissed.

Mr. A. J. Hammond, M. Am. Soc. C. E., has relinquished his position as City Engineer of South Bend, Ind., and has opened an office for general practice as a civil and hydraulic engineer at 207 Summers Bldg., South Bend, Ind.

Mr. Geo. E. Graham, Superintendent of Terminals of the Canadian Pacific Ry. at Fort William, Ont., has been transferred to a similar position at Vancouver, B. C. His place at Fort William will be taken by Mr. B. Armstrong.

Mr. C. J. Jeppeson, who has been associated for a number of years with the J. B. & J. M. Cornell Co., structural contractors, of New York City, as Chief Engineer, has opened an office as a consulting engineer at 118 East 28th St., New York City.

Mr. N. C. Hurst, who has been employed for the past three years with the William Tod Co., of Youngstown, Ohio, has recently joined the engineering staff of the Epping-Carpenter Co., of Pittsburg, Pa. Both of these companies are manufacturers of pumping machinery.

Mr. Winston Lee Winters and Mr. Frank J. Dove, civil engineers of Fort Smith, Ark., have united in partnership under the firm name of Winters & Dove, with office at 810 First National Bank Bldg., Fort Smith, Ark. They expect to give particular attention to municipal engineering.

Mr. Edwin J. Haddock, M. Am. Soc. M. E., formerly Chief Engineer of the chain department of the Jeffrey Manufacturing Co., has given up his office in Columbus, Ohio, to become Mechanical and Structural Engineer of the Tennessee Coal, Iron & R. R. Co. in the coal mining department, with office at Birmingham, Ala.

Mr. C. W. Kinney, Superintendent of the Auburn division of the Lehigh Valley R. R., has been appointed Superintendent of Transportation to succeed Mr. R. W. Baxter, resigned. Mr. C. J. Shea has been appointed to succeed Mr. Kinney as Superintendent of the Auburn division. Mr. J. N. Haines has been appointed Assistant Superintendent of the Buffalo division to succeed Mr. Shea.

Mr. Virgil G. Bogue, M. Am. Soc. C. E., has resigned his position as Vice-President and Chief Engineer of the Western Pacific Ry. and will return to New York City. His principal assistant, Mr. H. M. McCartney, has also resigned, but will remain in California. It is unofficially reported that Mr. T. J. Wyche, Division Engineer of the Western Pacific at Elko, Nev., will succeed Mr. Bogue as Chief Engineer.

Mr. Chas. H. Ackert has resigned his position of Vice-President and General Manager of the Southern Ry., effective Jan. 15, to become Vice-President in charge of maintenance and operation of the Chicago & Alton; Toledo, St. Louis & Western; Iowa Central and Minneapolis & St. Louis railways, commonly known as the Hawley lines. Mr. Ackert will remove his headquarters from Washington, D. C., to Chicago, Ill.

Mr. David W. Hays, an assistant engineer employed by the U. S. Reclamation Service on the Truckee-Carson irrigation project in Nevada, has resigned to engage in private practice at Reno, Nev. Before entering the Reclamation Service in 1903, he was an assistant hydrographer and engineer in the U. S. Geological Survey in stream measurement and surveys of canals and reservoirs.

Mr. Oliver D. Filley, Supervising Engineer for the War Department in the Philippine Islands, sailed for Manila from San Francisco Dec. 28 after a visit, on leave of absence, at his former home in Troy, N. Y. Mr. Filley went to the Philippines in 1903 as Supervisor of the Province of Leyte. His most important work during his six years in the archipelago has been the design and construction of the Governor Reynolds bridge in Albay Province.

Mr. J. W. Coyle, who was connected with the Best American Calorific Co. until its dissolution, has entered the service of the Rockwell Furnace Co. as a special salesman of oil and gas furnaces for railway work. Mr. Coyle was formerly Master Blacksmith of the Lehigh Valley R. R. at Wilkes-Barre, Pa., and was later in charge of the drop hammer and machine department of the Philadelphia & Reading Ry.'s forge shops at Reading, Pa.

Mr. A. B. Shepherd, Superintendent of the Bureau of Water of Pittsburg, Pa., has resigned to become Vice-President of the Monongahela Connecting R. R., effective Feb. 1. Mr. W. W. Willock, present Vice-President and General Manager of the Monongahela Connecting R. R., will devote his time after that date to the interests of the Jones & Laughlin Steel Co., owner of the railroad. Mr. Frank McCune, General Superintendent of the railroad, will be made General Manager.

Mr. Charles M. Hays, who has now formally come into office as President of the Grand Trunk Ry., has announced the appointment of E. H. Fitzhugh to be First Vice-President; Wm. Wainwright, Second Vice-President; M. M. Reynolds, Third Vice-President, and R. S. Logan, assistant to the President. Mr. Fitzhugh, Mr. Wainwright and Mr. Reynolds were formerly Third, Fourth and Fifth Vice-President, respectively. Mr. Logan was Mr. Hays' assistant while he was yet Second Vice-President and General Manager. Mr. Alfred W. Smithers, formerly First Vice-President, has been made chairman of the board of directors in London, England. Mr. Hays will himself retain the general managership.

Obituary.

John A. J. Shultz, President and founder of the Shultz Belting Co., died Dec. 24 at his home in St. Louis, Mo.

Harmanous L. Perrine, a mechanical engineer employed by the Scranton Steam Pump Co., died from pneumonia Jan. 1 at his residence in Allenhurst, N. J. He was born at Freehold, N. J., in 1846.

Jacobus Van der Hoek, whose death was noted in our last week's issue, was overcome by gas fumes, according to information received from a correspondent, while inspecting a tunnel 60 miles south of Wilkesbarre on the Lehigh Coal & Navigation Co.'s line between Mauch Chunk and Tamaqua, Pa. Earlier accounts stated that he was run down by a freight train.

Stephen Warner Baldwin, M. Am. Soc. M. E., who was for many years an officer of the Pennsylvania Steel Co., died Jan. 5 at Brookline, Mass. Mr. Baldwin was born at Baldwinsville, N. Y., in 1833, and began his business career at Lawrence, Mass. He was a Manager of the American Society of Mechanical Engineers in 1887-90 and a Vice-President of the society in 1890-'92.

Emil Strub, long known beyond the limits of Switzerland for his activities in the field of mountain railways, died at Zurich on Dec. 15, 1909, of heart failure, at the age of 51. He designed numerous cableway inclines and other mountain lines, but was best known as the inventor of the Strub rack rail. He was connected with the Swiss railway systems for ten years, but since 1898 had been in private practice.

Hon. G. Hilton Scribner died at Yonkers, N. Y., on Jan. 5, aged 78 years. Mr. Scribner was prominent during his long and active life in the financial and business circles of New York City. He was the President of some of the New York City street railway lines prior to the introduction of electric traction. He was Secretary of State of New York in the early '70's. Although educated as a lawyer, Mr. Scribner all his life was deeply interested in science. He held membership in many scientific organizations and contributed some notable papers to scientific publications. One of Mr. Scribner's sons, Mr. G. H. Scribner, is a well-known contractor of Chicago. A daughter of Mr. Scribner's is the wife of Mr. D. McN. Stauffer, M. Am. Soc. C. E., for many years editor of Engineering News. Another daughter married Mr. Edwin Hunt Frost, Assistant Manager of the Engineering News Pub. Co.

William C. Post, Vice-President and Treasurer of the firm of Post & McCord, structural iron dealers, of New York City, died Jan. 5 at Atlantic City, N. J.

Mr. Post was born Jan. 1, 1867, at Jersey City, and was the son of Andrew J. Post, the founder of Post & McCord, and the grandson of Simeon S. Post, one of the charter members of the American Society of Civil Engineers, and an engineer and bridge builder of considerable note in his time. William Post's early education was obtained in the public schools of Jersey City and the Preparatory School of Stevens Institute of Technology. He entered the Institute and graduated with the Class of '86. After graduation he immediately entered the employ of the firm of Post & McCord as a draftsman and rapidly rose to the position of Chief Draftsman, which position he held until the firm was merged with the American Bridge Co. in 1900. At that time he was appointed Contracting Manager, having entire charge of the vast business of this company in the vicinity of New York City. As evidence of his ability as an organizer, the drafting room and shop bill systems which he originated with Post & McCord are practically those in use by the American Bridge Co. at the present time. He left the American Bridge Co. in 1904 to assume the position of Vice-President and Treasurer of the newly organized Post & McCord, Incorporated, which he held until the time of his death.

Mr. Post had a large circle of friends in the building trades and was a prominent member of several organizations connected therewith, holding the position up to last October of President of the Building Trades Credit Association. At the time of his death, he was a member of the Board of Governors of the Building Trades Employers' Association, and represented his company in the Iron League Erectors' Association of New York and the National Erectors' Association. He was formerly on the Executive Committee of the Stevens Alumni Association and was afterwards elected successively Vice-President, President and Alumni Trustee. Mr. Post was a member of the Engineers' Club of New York City and of numerous other clubs in Pennsylvania, South Carolina, Canada and New Jersey.

His funeral services were held at his late residence in Englewood, N. J., on Friday, Jan. 7, and the interment was at the Brookside Cemetery, Englewood.

Col. George Earl Church, M. Am. Soc. C. E., died Jan. 5 at London, Eng. He was born at New Bedford, Mass., Dec. 7, 1835, and was educated at the public schools of Providence, R. I., and by private tutors. Col. Church's mother had removed with him to Providence in his eighth year, after the death of his father. He was employed for some time on state surveys for the map of Massachusetts and afterward entered the service of a railway in New Jersey. He next obtained a position as Assistant Engineer of a railway under construction in Iowa. After a short term as Resident Engineer of the Hoosac Tunnel, he returned to Iowa, at the time of the temporary suspension of the Hoosac Tunnel work, as Chief Assistant Engineer on the location of a railway in that state. At this time he was only 21 years old.

The railway location work was abandoned on account of the financial crisis of 1857 and Col. Church went to South America to take charge, as Chief Engineer, of a projected railway in the Argentine Republic. This work also had to be given up, however, on account of the unsettled condition of the country, but in 1858 Col. Church was appointed a member of a scientific expedition to explore the southwestern Argentine frontier. In 1860 he surveyed and located the Great Northern Ry. of Buenos Ayres but left that city at the outbreak of the Civil War to return to New England and was commissioned as a captain in the Seventh Rhode Island Infantry. He served in the Army of the Potomac in 1862-5, attaining successively the ranks of lieutenant-colonel, colonel and brigade commander.

Immediately after the close of the war, he was made Chief Engineer in charge of construction of the Providence, Warren & Fall River Ry., but he soon re-entered the army to serve in Mexico in the last two campaigns against the Maximilian Empire in 1866-7. During this period he acted also as a war correspondent for the New York "Herald" and he afterward joined for a time the "Herald's" editorial staff. South America was the principal scene of the next thirty years of Col. Church's life work. Ten or twelve years he spent in extensive explorations of the interior of South America. His explorations included a descent of the River Amazon from its Bolivian source and he subsequently engaged in an effort to open Bolivia to commerce via that river. He served as U. S. Commissioner to Ecuador in 1880 and made a voyage to Costa Rica in 1895 to report on a railway in that country. In Engineering News for Nov. 12, 1908, p. 531, a portrait of Col. Church was published in connection with an account of a dinner given in his honor in Philadelphia by the Madeira and Mamoré Association. Eleven years of his life, from 1868 to '79, were spent in promoting the Madeira and Mamoré Ry. in Brazil.

Col. Church represented the American Society of Civil Engineers at the London Congress of Hygiene and Demography in 1891. He was President of the Geographical Section of the British Association in 1898 and for four years was Vice-President of the Royal Geographical Society. For many years before his death he was a resident of London, where he engaged in the development and promotion of Canadian railway enterprises.

Engineering Societies.

COMING MEETINGS.

AMERICAN SOCIETY OF HEATING AND VENTILATING ENGINEERS.
Jan. 18-20. Annual meeting at New York City. Secy., W. M. Mackay, P. O. Box 1818, New York City.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
Jan. 19-20. Annual meeting at New York City. Secy., Chas. W. Hunt, 220 West 57th St., New York City.

AMERICAN SOCIETY OF INSPECTORS OF PLUMBING AND SANITARY ENGINEERS.
Jan. 20-22. Annual convention at Trenton, N. J. Secy., C. S. McCosker, Mobile, Ala.

CANADIAN SOCIETY OF CIVIL ENGINEERS.
Jan. 25-27. Annual meeting at Ottawa, Ont. Secy., C. H. McLeod, 413 Dorchester St. West, Montreal, Que.

ILLINOIS SOCIETY OF ENGINEERS AND SURVEYORS.
Jan. 26-28. Annual meeting at Cairo, Ill. Secy., E. E. R. Tratman, 1636 Monadnock Block, Chicago, Ill.

NATIONAL BRICK MANUFACTURERS' ASSOCIATION.
Feb. 7-12. Annual convention at Pittsburg, Pa. Secy., T. A. Randall, Indianapolis, Ind.

CONNECTICUT SOCIETY OF CIVIL ENGINEERS.
Feb. 8. Annual meeting at New Haven, Conn. Secy., J. Frederick Jackson, Box 1304, New Haven, Conn.

OHIO ENGINEERING SOCIETY.
Feb. 8-10. Annual meeting at Columbus, Ohio. Secy., C. J. Knisely, New Philadelphia, Ohio.

NEW ENGLAND ASSOCIATION OF GAS ENGINEERS.
Feb. 16. Annual meeting at Boston, Mass. Secy., N. W. Gifford, 26 Central Square, E. Boston, Mass.

IOWA ENGINEERING SOCIETY.
Feb. 16-17. Annual convention at Cedar Rapids, Iowa. Secy., A. H. Ford, Iowa City, Iowa.

NORTHWESTERN CEMENT PRODUCTS ASSOCIATION.
Feb. 18-21. Annual convention at Chicago, Ill. Chairman, notification committee, O. U. Miracle, Minneapolis, Minn.

NATIONAL ASSOCIATION OF CEMENT USERS.
Feb. 21-26. Annual convention at Chicago, Ill. Pres., Richard L. Humphrey, Harrison Bldg., Philadelphia, Pa.

AMERICAN SOCIETY OF ENGINEERING CONTRACTORS.
Feb. 24-26. Annual meeting at Chicago, Ill. Secy., D. J. Hauer, 721 Park Row Bldg., New York City.

AMERICAN RAILWAY ENGINEERING AND MAINTENANCE OF WAY ASSOCIATION.
March 15-17. Annual convention at Chicago, Ill. Secy., E. H. Fritch, 962 Monadnock Block, Chicago, Ill.

WESTERN SOCIETY OF ENGINEERS.—The election of officers for 1910 has resulted as follows: President, T. W. Alvord; Vice-Presidents, O. P. Chamberlain, A. Bement and W. K. Hatt; Treasurer, A. Reichmann. The Secretary is J. H. Warder, 1735 Monadnock Block, Chicago.

ILLINOIS SOCIETY OF ENGINEERS AND SURVEYORS.—The 25th annual meeting will be held at the Armory Hall, Cairo, Ill., Jan. 26 to 28; the hotel headquarters will be at the Halliday Hotel. The principal papers on the program are as follows: Jan. 26, "The Relation of Typhoid Fever Death Rate to the Water-Supplies in Illinois," E. Bartow; "Municipal Engineering Work at Cairo," W. B. Thistlewood; "Chicago's Sidewalk Problem," N. E. Murray; "Materials and Workmanship in Street Paving," W. W. Marr; "Municipal Ownership of Public Utilities," J. W. Dappert; "Sewage Disposal Plants for Small Institutions," G. C. Habermeyer; "Recent Examples of Sewage Purification," W. S. Shields; "Reinforced-Concrete Construction," A. J. Widmer; "Reinforced-Concrete Aqueducts," H. E. Reeves; "Tests of Road Materials," R. B. Slippy.

Jan. 27, "The Drainage and Protection of Lands Subject to Overflow," C. W. Brown; "The Legal Liabilities of Levee and Drainage Districts," J. V. Waddell; "The Tallahatchie Drainage District," A. L. Dabney; "The Maintenance of Open Ditches," E. H. Cowan; "Water Supply and Drainage Investigations in Illinois," R. I. Randolph; "Observations of Engineering in Europe," F. O. Dufout; "Developments in Road Construction," A. N. Johnson.

Jan. 28, "The County Surveyor," P. T. Hicks; "The Cairo & Thebes Ry.," G. F. Dewey; "Steel and Concrete Bridge Work on the Illinois Central Ry.," R. E. Gaut; "Monazite and Its Uses," S. K. Atkinson; "Railway Engineering at the University of Illinois," Prof. S. S. Roberts; "Mining Engineering at the University of Illinois," Prof. H. H. Stoek; "Instruction in Hydraulics and Water Supply Engineering," Prof. G. C. Habermeyer; "Instruction in Road and Pavement Construction at the University of Illinois," R. B. Slippy; "The St. Louis and Cairo Division of the Deep Waterway," J. W. Woermann.